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UNITED STATES
COMMITTEE ON LARGE DAMS

Dam Safety and Rehabilitation

Fourth Annual USCOLD Lecture

January 24, 1984

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USCOLD LECTURES

1981	James Bay Hydro Development
1982	Tarbella Dam Project, Pakistan
1983	Bath County Hydroelectric Pumped-Storage Project
1984	Dam Safety and Rehabilitation

1984 SPEAKERS

David L. Hinchliff –	Hydraulic Design and Application of Labyrinth Spillways
Edward W. Gray, Jr. –	Fuse Plug Embankments in Auxiliary Spillways Developing Design Guidelines and Parameters
Lloyd O. Timblin, Jr. –	Flexible Membrane Emergency Spillway
C. O. Duster –	Dam Crest Raising With Reinforced Earth Retaining Walls
L.R. Carpenter –	Automatic Monitoring of Structural Behavior Instruments for Reclamation Concrete Dams
Raymond G. Acciardi –	Improvements to USBR Pinhole Test Equipment - Design and Test Result Evaluation
Douglas Craft –	Chemical Tests for Dispersive Soils - Problems and Recent Research
Luther Davidson –	Electronic Monitoring of Cement Pressure Grouting at Ridgway Dam

ERRATA

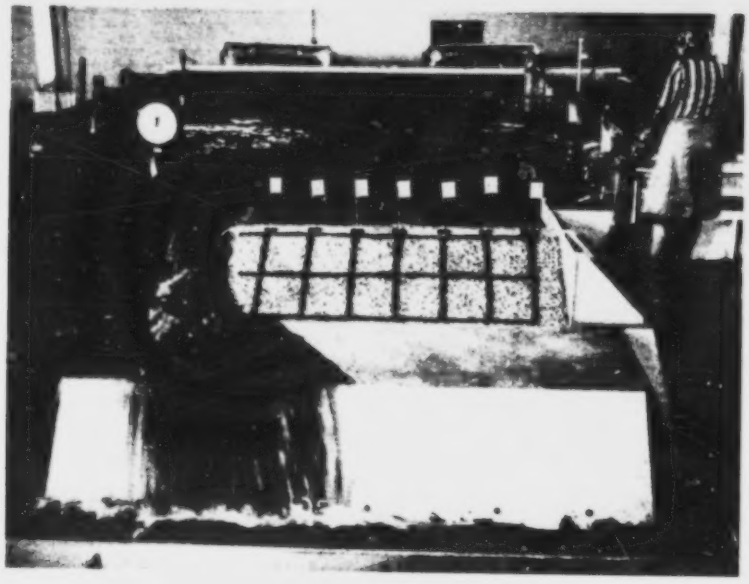
Fourth Annual USCOLD Lecture

Fuse Plug Embankments in Auxiliary Spillways Developing Design Guidelines and Parameters

Page 2-v: The symbol for boundary or grain Reynolds number should read "R*."

Page 2-20: Equation (9) should read " $(\frac{\gamma L}{E})_m = (\frac{\gamma L}{E})_p$."

Page 2-27: Figure 2-26(b) is not the correct photograph. The correct photograph is shown below.



Page 2-36: The eighth column heading in table 2-2 should read "p/H."

III

UNITED STATES
COMMITTEE ON LARGE DAMS

Dam Safety and Rehabilitation

Fourth Annual USCOLD Lecture
Phoenix, Arizona

January 24, 1984

Presented By

Bureau of Reclamation
U. S. Department of the Interior

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FOREWORD

USCOLD (United States Committee on Large Dams) has a strong commitment to dam safety, which is reflected in the topic for this Fourth Annual USCOLD Lecture, "Dam Safety and Rehabilitation."

In July 1966 USCOLD published "Supervision of Dams by State Authorities," because it was "concerned with the design, construction, and maintenance of dams to the end that good, safe, economical dams will be constructed and that the maintenance and operation of such structures will be safe and adequate."

In 1970 USCOLD submitted to State and Federal authorities the "Model Law for State Supervision of Safety of Dams and Reservoirs" to provide the States with guidelines for establishing a dam safety program for new and existing dams.

The American Society of Civil Engineers and USCOLD jointly published "Lessons From Dam Incidents, USA," in 1975. This book discussed several hundred U.S. dams that had experienced problems and the remedial measures taken to correct the deficiencies.

In January 1982 USCOLD distributed to all 50 States a resolution concerning "Safety of Dams in the United States." In this resolution, USCOLD stated its position that "a large number of dams in the United States do not meet current minimum acceptable safety criteria, that each State Government should actively pursue the supervision of dam safety for the protection of its population, and that the Federal Government should assist the States to develop their own capability in supervising dam safety."

In April 1982 USCOLD published "Dam Safety Practices and Concerns in the United States," which included papers on the current progress toward improving dam safety and other related topics. Furthermore, USCOLD formed its Dam Safety Committee, in 1982, to place additional emphasis on safety-related issues. All of these actions express USCOLD's intense interest in constructing and maintaining safe dams in the United States.

The USBR (Bureau of Reclamation) shares USCOLD's strong commitment to dam safety. The USBR is responsible for the safe operation of more than 275 dams in the Western United States. These dams, some of which date back to the early 1900's, present a wide variety of safety-related problems. A few of these problems and their solutions are discussed in this Fourth Annual USCOLD Lecture.

The evaluation of new hydrologic and meteorologic data has resulted in the need for increased spillway capacity at many USBR dams; but modification of an existing dam can be a difficult engineering task, requiring

creative and innovative solutions. Several techniques for accommodating new design floods are presented in this lecture. These techniques include installation of a labyrinth spillway, a flexible membrane spillway, and a fuse-plug spillway as well as the use of reinforced earth to provide additional freeboard capacity.

Dam safety awareness has also renewed interest in the timely monitoring of instrumentation at existing dams. A report is presented on the USBR's recently developed automatic system for monitoring instrumentation at several of its dams.

Dispersive soils were responsible for many dam failures during the 1960's and 70's. The early testing criteria developed by the Soil Conservation Service and Dr. James Sherard, et al., have been reanalyzed. The alternative testing procedures which have been developed are presented.

A dam currently under construction in western Colorado employed the first application in the United States of electronic monitoring of cement pressure grouting for a dam foundation. A paper is presented on the monitoring methods used, the problems encountered, and the solutions applied.

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HYDRAULIC DESIGN AND APPLICATION OF LABYRINTH SPILLWAYS

by

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January 1984

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Denver, Colorado

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ABSTRACT

A significant portion of the work performed by the Bureau of Reclamation deals with dam safety and rehabilitation. Inadequate spillway capacity is one of the primary reasons some Bureau dams require rehabilitation and modification. With advances in the fields of hydrology and meteorology and an increased streamflow and runoff data base, the probable maximum flood a dam must safely withstand may increase substantially over the original design flood. This is especially true for older dams. If design analysis indicates that a spillway may not be adequate to safely pass the updated flood, resulting in overtopping the dam and possible failure, modifications to the spillway must be made.

The Bureau is also involved in enlarging dams and reservoirs to meet increasing downstream water demands, to provide additional flood control capacity in reservoirs, and to develop greater hydroelectric generation capabilities. One of the major difficulties in raising a dam is modifying the spillway to function adequately at higher reservoir levels.

An alternative that should be considered for these modification needs is the use of a labyrinth spillway. The Bureau and other engineering organizations are finding that labyrinth spillways are particularly well suited for rehabilitation of existing spillway structures because the developed crest length can be greatly increased for a given width. This increased crest length allows passage of a greater design flood than the existing structure. A free overflow labyrinth spillway provides reservoir storage capacity equal to the traditional gated structure, which requires manual or mechanical operation. In addition, labyrinth structures may be built economically provided an adequate foundation is available and the structure does not exceed certain established limitations.

LABYRINTH SPILLWAY PARAMETERS AND FLOW DESCRIPTION

The labyrinth spillway is defined by the parameters shown on figure 1-1. The values of these parameters are chosen to accommodate site geometry and to provide optimum hydraulic performance. The plan geometry is defined by the parameters of length, l, width per cycle, w, sidewall angle, α, and the number of cycles, n. The length and width per cycle may be combined to form the dimensionless length magnification, l/w. The vertical geometry of the labyrinth is described by the spillway height, P, and the vertical aspect ratio, w/P.

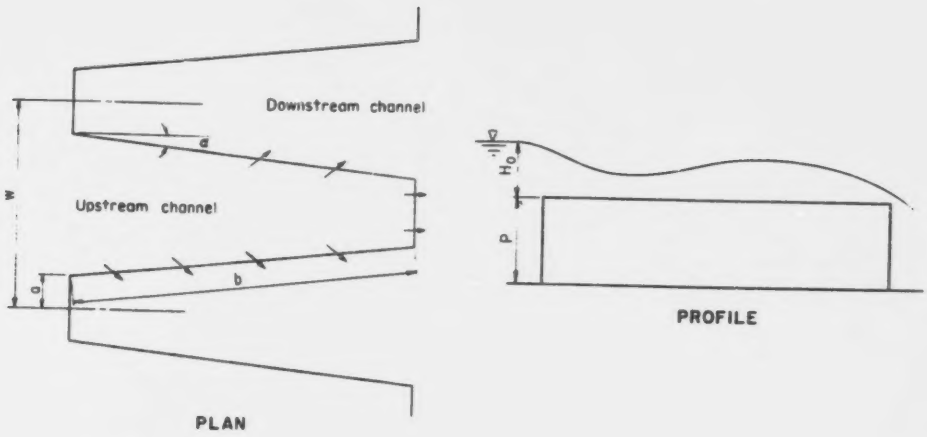
The performance of the labyrinth spillway is directly related to the discharge, Q_N, passed by a linear weir of width, W, equal to the total width occupied by all the labyrinth spillway cycles. Therefore, two analyses must be performed - one for the linear weir discharge and one for the labyrinth weir discharge. The labyrinth length required to pass the design discharge is then determined from design curves that show the labyrinth to linear discharge ratio, Q_L/Q_N, and the head to crest height ratio, H/P.

Ideally, the discharge passing over the labyrinth spillway should increase in direct proportion to an increase in the crest length. For instance, a length magnification of three should allow passage of a discharge three times as great. However, this is only the case for spillways with low head to crest height ratios, because the spillway efficiency decreases as the head increases. In addition, this efficiency loss is greater and occurs more rapidly with greater length magnifications.

Description of the flow over a labyrinth spillway is complicated and will be explained in terms of the reservoir head, the local head present in the upstream channels, flow over the weir, and the tailwater depth in the downstream channels. The flow over a labyrinth spillway passes through three basic phases: subatmospheric pressure under the nappe, an aerated nappe, and a nonaerated solid water nappe. These flow phases occur as the head to crest height ratio increases from very low values (less than 0.15) to the maximum design value. These changes in the flow conditions are clearly seen in the behavior of the discharge coefficient and a discontinuity, or "hump," in the length magnification curve.

With small heads over the spillway, the flow behaves almost ideally with an almost negligible head difference between the upstream reservoir and the spillway channels. However, with low flows, subatmospheric pressures under the nappe cause the nappe to cling to the downstream face of the spillway. This low flow condition (fig. 1-2) produces an increase in the discharge coefficient, but may also cause structural problems.

Median range discharges and head to crest height ratios produce a noticeable drop in head as the flow from the reservoir enters the upstream channels. Farther into the channels the water surface rises again, but



LEGEND

a	= Half length of labyrinth apex
b	= Length of labyrinth wall
H	= Total upstream head over crest (less than H_o)
H_o	= Design head
l	= Developed length of one labyrinth cycle = 4a + 2b
L	= Total developed length of spillway
l/w	= Length magnification
n	= Number of spillway cycles in plan
P	= Spillway height (crest height)
Q_L	= Discharge over labyrinth spillway
Q_N	= Discharge over linear spillway
Q_L/Q_N	= Flow magnification (measure of spillway performance)
W	= Width of linear spillway
w	= Width of one labyrinth spillway cycle
w/P	= Vertical aspect ratio
α	= Angle of sidewalls to main flow direction

Figure 1-1. - General plan and section of labyrinth spillway with definition of parameters.

Figure 1-2. - Labyrinth low flow condition.

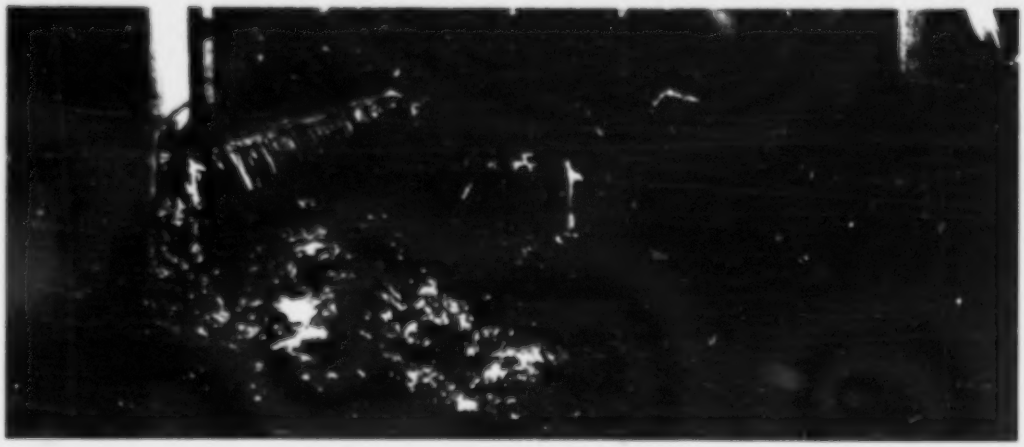


Figure 1-3. - Labyrinth medium flow condition.

never returns to the original reservoir level. In this median head to crest height range the spillway nappe alternates between being aerated and nonaerated. As the head increases, the nappe becomes aerated and springs free of the downstream face, producing the most desired operating condition. However, with a further increase in head, the nappe thickens and begins closing off the area between the nappe and the downstream spillway face. At this point, the flow alternates between being aerated and nonaerated, with air being drawn under the nappe at the downstream apexes of the spillway and intermittently moving upstream (fig. 1-3). This unstable flow condition produces a discontinuity in the discharge coefficient curve.

The final flow condition consists of the higher head to crest height ratios ($H/P \leq 0.4$) and produces an even greater upstream head loss as the flow enters the spillway channels. Flow over the spillway is in the form of a solid nonaerated nappe. The thickness of the nappe and the tailwater height do not permit air to be drawn under the nappe (fig. 1-4). Eventually, as the head increases, the spillway becomes submerged producing extremely inefficient spillway operation.

CASE STUDY - UTE DAM

To illustrate the design and construction considerations involved with labyrinth spillways, the Bureau's experiences with the modifications to Ute Dam in New Mexico are discussed in the following sections.

Ute Dam, completed in 1963, is owned and operated by the NMISC (New Mexico Interstate Stream Commission). The dam is located on the Canadian River in east-central New Mexico, near the community of Logan. The existing facility consists of a zoned embankment main dam with a maximum height of approximately 120 feet; an ungated ogee-type concrete spillway located to the left of the main dam with a crest length, W, of 840 feet; and an embankment dike located to the left of the spillway with a maximum height of approximately 25 feet (fig. 1-5).

The dam as originally constructed did not provide sufficient reservoir capacity to permit the State to use its full storage allotment, as agreed in the Canadian River Compact. The NMISC requested that the Bureau undertake the investigation, design, specifications, and construction of the addition of 27-foot-high spillway gates, which would increase the reservoir to its desired capacity. The Bureau prepared appraisal designs and estimates for several types of gated structures having a minimum field cost of approximately $34 million (based on November 1980 unit prices). This cost was unacceptable to the NMISC. The Bureau then prepared several designs and estimates for ungated alternatives that provided the necessary normal reservoir capacity and limited the maximum water surface elevation during floods to prevent the inundation of homes around the reservoir. The most economical alternative was a labyrinth spillway combined with raising the dam for an estimated cost of $10 million. In 1981, the NMISC accepted the

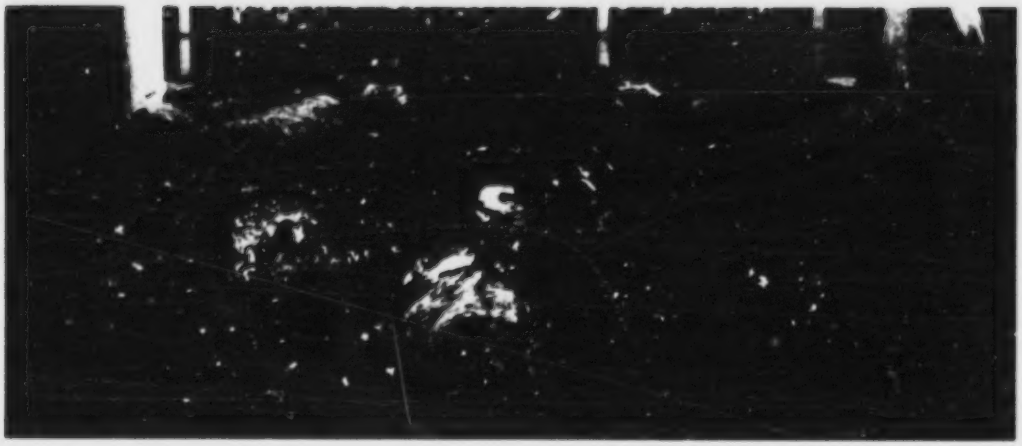


Figure 1-4. - Labyrinth high flow condition.

Figure 1-5. - Ute Dam near Logan, New Mexico, before modification.

labyrinth spillway concept and provided funds for laboratory investigations and the preparation of the final design and specifications.

Hydraulic Model Studies

Hydraulic model studies were initiated to extrapolate existing design curves [5]* for application to the Ute Dam labyrinth spillway. These tests included flume testing of two-cycle labyrinth weir sectional models and two 1:80 scale models of the proposed Ute spillway designs [6, 7].

The design criteria for the Ute labyrinth spillway were:

H_o = 19 feet	W = 840 feet
P = 30 feet	H_o/P = 0.63
Q_L = 590,000 ft^3/s	

These criteria were based on the existing site geometry and the IDF (inflow design flood). The remaining parameters were determined during the design process and investigated during the model study. For the initial 10-cycle labyrinth spillway model, these parameters were:

w = 84 feet	a = 3 feet
l/w = 2.74	b = 109.1 feet
w/P = 2.8	L = 2,300 feet
α = 19°15'55"	

Model testing of this labyrinth spillway - based on design curves published by Hay and Taylor [5] - showed the design discharge could not be passed by the spillway within the stipulated design head of 19 feet. The reservoir head reached 22.6 feet before passing the required maximum discharge. This was a result of the large head to crest height ratio and an inadequate labyrinth crest length. The crest length was inadequate because of the characteristics of the flow over the labyrinth. Further details of this 10-cycle labyrinth spillway may be found in [6].

Because the 10-cycle spillway did not pass the required discharge within the reservoir head limitation, another model spillway of longer crest length was designed and tested. This spillway design was based on the results of the 10-cycle spillway tests and additional flume testing. The most economical design, given the new longer crest length, required 14 cycles. The other labyrinth spillway parameters for this design were:

w = 60 feet	a = 3 feet
l/w = 4.0	b = 114 feet
w/P = 2.0	L = 3,360 feet
α = 12°8'15"	

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* Numbers in brackets refer to entries in the Bibliography.

Comparing these parameters with those of the 10-cycle spillway shows the difference between the crest lengths and length magnifications in the two designs.

The 14-cycle spillway passed the required maximum discharge at 19 feet of head (fig. 1-6). In addition to the labyrinth spillway shape, other aspects of labyrinth spillway operation determined with this model included the effect of nappe interference, impact pressures in the downstream channels, water surface profiles in the upstream channels, and low flow conditions. These aspects will be discussed in the section dealing with general design guidelines.

Structural Analysis and Design

Once the hydraulic design and model studies were completed, the Ute Dam labyrinth spillway was analyzed for stability and structural integrity. The labyrinth spillway was analyzed as a series of 14 V-shaped cycles. Thirteen of the cycles are monolithic and separated by contraction joints. The remaining cycle consists of two monolithic half cycles - one at each end of the spillway.

The stability analysis of a typical labyrinth cycle included the investigation of overturning, sliding, and foundation bearing pressures when the cycle was subjected to the following loads:

Normal load - normal water surface (elevation 3787), no tailwater, uplift assumed to be full head under area of base upstream of the wall.

Extreme load - maximum water surface (elevation 3806), tailwater height of 15 feet, uplift varying from full head at the upstream edge of the labyrinth to tailwater head at the downstream edge.

Analyses showed that a typical full cycle was stable against overturning, but required a 5-foot-deep key trench to provide an adequate factor of safety against sliding, when subjected to the extreme load. The foundation bearing pressure was acceptable for both loading conditions. The analysis on an end half cycle of the labyrinth showed that it was not stable against overturning. To make the half cycle stable, an anchor block was attached to the existing spillway end wall. The anchor block resisted upward movement of the labyrinth base slab and transferred the load to the existing wall. However, this additional upward load on the existing wall lowered the wall's factor of safety against sliding, which made it unstable when subjected to the load from the maximum reservoir water surface elevation. Therefore, to make the existing wall and the labyrinth half cycle stable against sliding, a key trench, parallel to the existing wall, was added to the base of the half cycle. The anchor block and key trench allowed the existing end wall and the labyrinth half cycle to act as a unit. A contraction joint was placed between the two to ensure that compressive loads would be transmitted from the existing wall to the labyrinth, and to prevent tensile loads from being transmitted from the labyrinth to the wall.

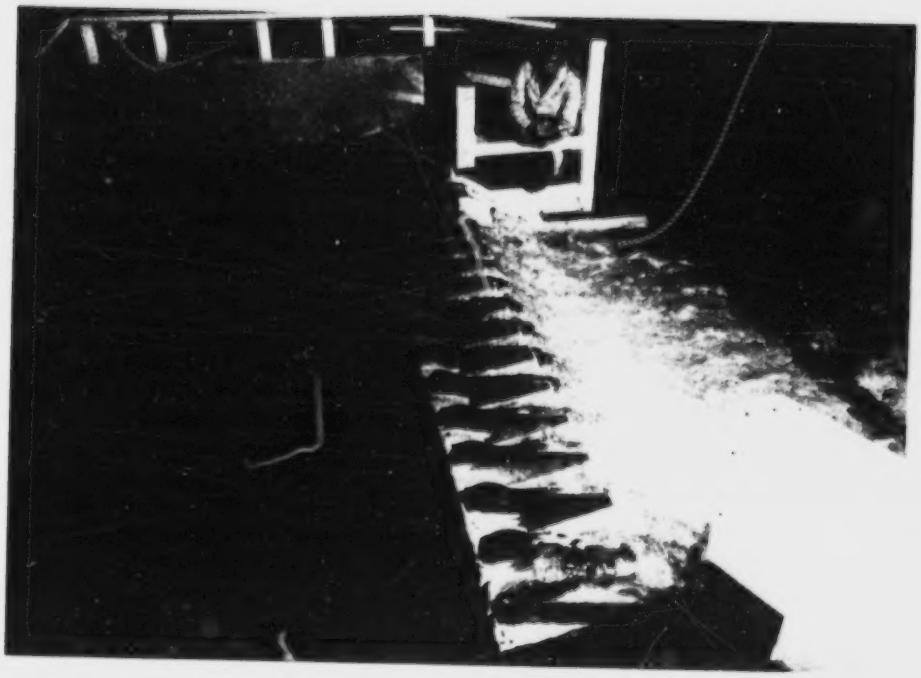


Figure 1-6. - 1:80 scale model of Ute Dam labyrinth spillway.

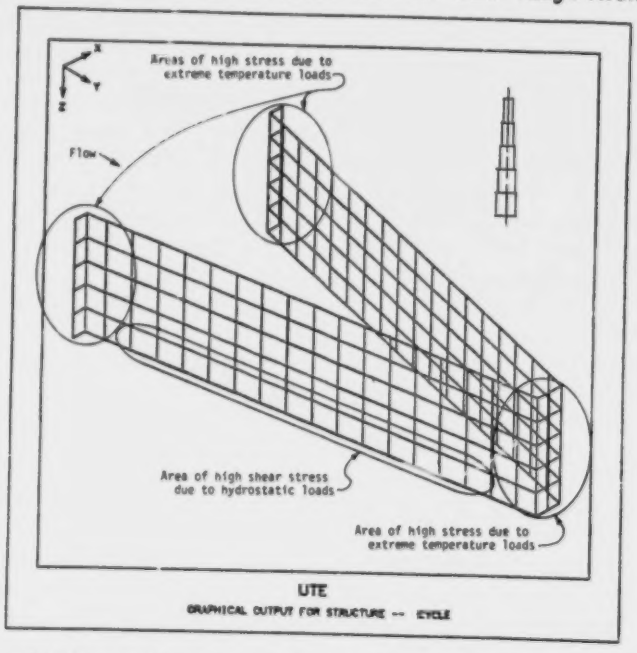

Areas of high stress due to extreme temperature loads
Flow
Area of high shear stress due to hydrostatic loads
Area of high stress due to extreme temperature loads
UTE
GRAPHICAL OUTPUT FOR STRUCTURE -- CYCLE

Figure 1-7. - Finite element model of the wall for the Ute labyrinth spillway.

Once the stability of the spillway was ensured, the individual components were sized and the reinforcement designed. The structural analysis was made on both a typical full and half cycle using a finite element indeterminate structural analysis computer program. Separate analyses were made for the labyrinth wall and base slab because of limitations in the program's ability to determine the number of unknowns. Sixteen different factored load combinations were applied to the wall of the labyrinth using various boundary conditions. The maximum stresses computed for the various elements of the computer model were used for determining the reinforcement required for that element.

The structural analysis showed that the high stresses were located in areas around the apexes of the labyrinth (fig. 1-7). These high stresses were primarily caused by the extreme temperature loads that developed from the large seasonal temperature variations typical for this vicinity. The apexes had to be stiffened by increasing the thickness of the concrete, and heavily reinforced to resist the high bending moments, tensile stresses, and shear stresses that developed. The large hydrostatic loads caused by the height of the wall and the depth of overtopping of the labyrinth required significant amounts of reinforcement for all other areas of the labyrinth wall as well.

The analysis of the half cycle wall indicated deflections at the downstream apex where the labyrinth meets the existing spillway end wall were too large to ensure watertightness. To keep the wall watertight without having to depend on the bond between the labyrinth and the existing wall, an anchor block was placed on the existing wall downstream of the labyrinth. A waterstop was installed between the labyrinth and the anchor block, and an expansion joint was installed to allow the labyrinth wall to deflect without transferring shear and tensile loads to the existing wall.

For the base slab analysis, loads from the wall were applied along a set of points where the centerline of the wall meets the base. These loads were determined by analyzing the wall, which was assumed to be fixed at the base. This produced a set of reactions that was then changed into loads to be applied to the base slab. Along with loads from the wall, additional loads such as the weight of the slab, the weight of water on the slab, temperature loads and uplift were applied to the base of the labyrinth under various boundary conditions and load combinations.

For the typical cycle, areas near the center of the base showed upward deflections as high as three-fourths of an inch when subjected to extreme load combinations (fig. 1-8). While the deflections did not seem excessive, a check was made to determine if anchor bars could be used to hold down the base. The results showed that the restraint of the anchor bars caused higher stresses in the concrete and that stresses in the different anchor patterns were usually concentrated on a few bars. This indicated that a progressive failure of anchor bars could occur. It was decided to let the base deflect and redistribute the stresses. As with the wall, the base slab was heavily reinforced to resist the high

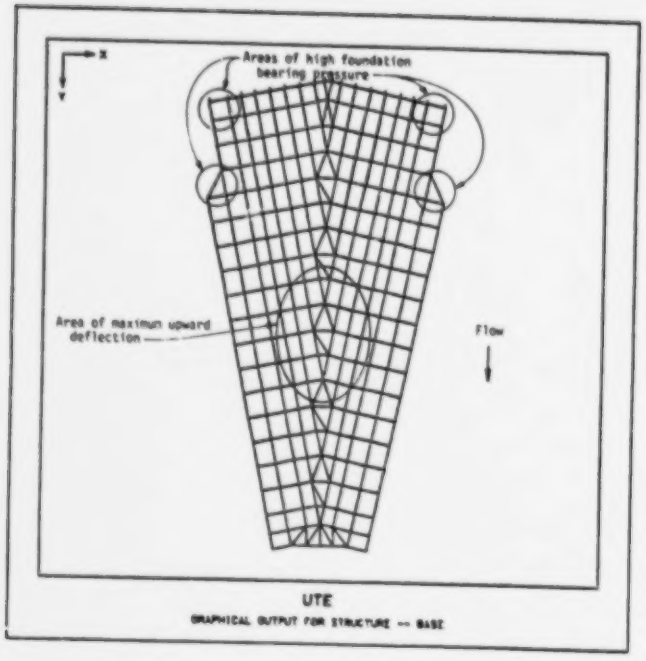
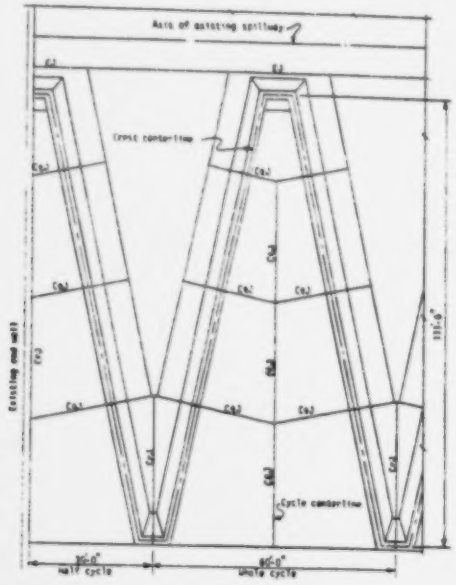


Figure 1-8. - Finite element model of the base slab for the Ute labyrinth spillway.



CtJ = Control joint (no bond, fully reinforced)
CrJ = Contraction joint (no bond, no reinforcement)
EJ = Expansion joint (no bond, joint filler)

Figure 1-9. - Joint layout for labyrinth spillway.

bending moments, tensile stresses, and shear stresses that developed from the various load combinations applied to the computer model.

In the design of the labyrinth spillway, four types of joints were used: contraction joints, control joints, construction joints, and expansion joints (fig. 1-9). The configuration of the labyrinth base slab permitted placement of contraction joints at the narrowest sections of the slab. This allowed each cycle of the spillway to act monolithically, with the contraction joints having unbonded surfaces and no reinforcement. The location of the contraction joints at the downstream apexes of the wall ensured that hydrostatic forces would tend to hold the wall joints closed.

Although the walls and base slabs were heavily reinforced, random cracking was possible throughout the structure due to its large size and the high stresses involved. Therefore, control joints were provided to concentrate cracking at predetermined locations. The joints were designed to have surfaces that were unbonded but fully reinforced. In addition, chamfers were provided at the surface of each joint. To keep each joint watertight after a crack had formed, waterstop was installed and a polysulfide sealant was applied to the upstream face of the joint.

Horizontal construction joints were placed in the wall of the labyrinth to allow for the placement of concrete in three 10-foot-high lifts. Because these joints were designed to be fully bonded and reinforced, they were not a factor in the design of the structure. Construction joints were not included in the base slab because the control joints provided satisfactory concrete placement dimensions.

To prevent the labyrinth spillway from transferring loads to the existing ogee crest structure, a 1-inch sponge-rubber-filled expansion joint was placed between the base slab and the existing crest structure.

Construction

Construction on the labyrinth spillway at Ute Dam began in November 1982. After a short period of mobilization, the contractor, KNC, Inc., of Albuquerque, New Mexico, began excavation for the labyrinth spillway foundation. A Roto-Mill profiler was used for excavating the sandstone to a uniform elevation of 3753.0 (fig. 1-10). The machine, used mainly in highway construction, had a rotating drum with carbide cutting teeth capable of removing approximately 3 inches per pass. A power broom was then driven over the excavated area to remove loose sand and clean the foundation surface producing a smooth, clean surface at the desired grade (fig. 1-11). A few areas of clay seams and fractured sandstone were encountered. These required overexcavation and backfilling with concrete. The 5-foot-wide key trench for the labyrinth base slab was excavated in two passes - each 2 feet wide - by a trenching machine. A backhoe excavated the remaining rock left in the trench (fig. 1-12).

Figure 1-10. - Roto-Mill profiler used for excavation of the foundation for the labyrinth spillway.

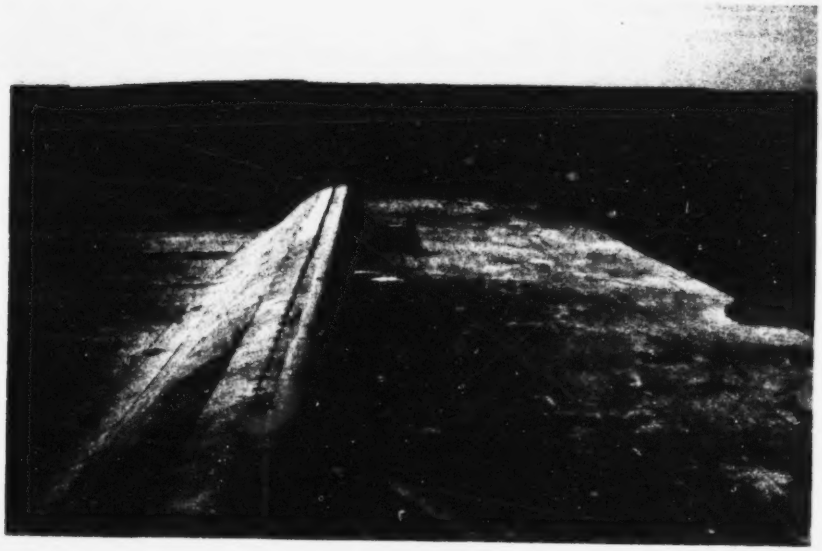


Figure 1-11. - Excavated surface for labyrinth spillway.

Figure 1-12. - Excavation of key trench.

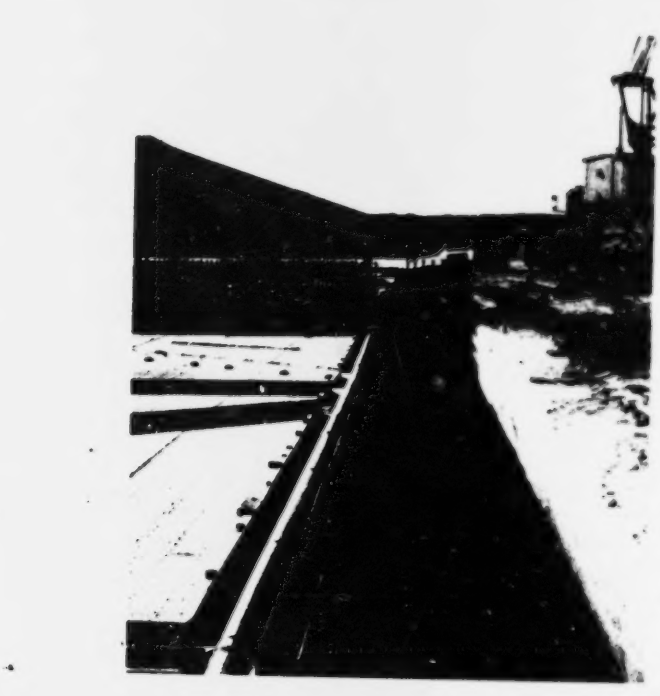


Figure 1-13. - Four-inch-diameter split drains placed on excavated surface before concrete encasement.

After excavation was completed, a series of 4-inch-diameter split drain pipes was installed on the foundation surface to intercept seepage and reduce uplift pressures on the base of the labyrinth (fig. 1-13). These split drains were encased in concrete to prevent them from being damaged during construction of the base slabs. Water collected by the split drains is carried downstream of the labyrinth and passes through the existing ogee in a series of holes drilled horizontally through the crest. To prevent excessive uplift pressures from developing beneath the existing crest structure, a line of 70-foot-deep relief wells was drilled immediately upstream of the crest. These wells were cased with slotted PVC (polyvinyl chloride) pipe and capped with a flap valve to prevent debris from plugging the hole.

Forms for the base slab were then constructed and reinforcement installed. Because the labyrinth is a cantilever-type structure, most of the reinforcement for the wall had to be embedded in the base slab before the concrete for the base was placed. This created difficulties in placing the large amounts of reinforcement required and in supporting the steel for the walls of the labyrinth. Forming the control joints within the base slab was also difficult because of the number of reinforcing bars that had to pass through the joint and the installation of PVC waterstops along the joint.

The center cycle of the labyrinth was the first to be constructed. Initially, work proceeded slowly as the contractor developed efficient methods of building forms and installing and supporting reinforcement, and as the steel supplier improved the steel cutting and bending operations. The pace of construction increased rapidly as additional cycles were constructed. Figures 1-14 and 1-15 show how construction of the cycles progressed.

Concrete, with a design strength of 5,000 lb/in^2 at 90 days, was placed for the base slab at each cycle in seven different sections, each delineated by control joints. Concrete for the walls was placed in 10-foot-high lifts, also delineated by control joints. The Bureau required that no concrete be placed in immediate sections of the labyrinth base or wall until the abutting concrete had been in place for at least 7 days. This was done to ensure that the concrete had completed expansion due to the heat of hydration and would be contracted to its final dimensions, providing tight joints and minimizing the possibility of seepage through the structure. This 7-day requirement has complicated placement schedules and forced the contractor to work on several cycles at a time. Yet, even with these scheduling complications and other construction problems, completion is expected in January 1984, before the contract completion date in May 1984.

Some of the major quantities of materials required for the construction of the labyrinth spillway at Ute Dam are:

Excavation - over 5,000 yd^3
Concrete in walls - over 13,000 yd^3

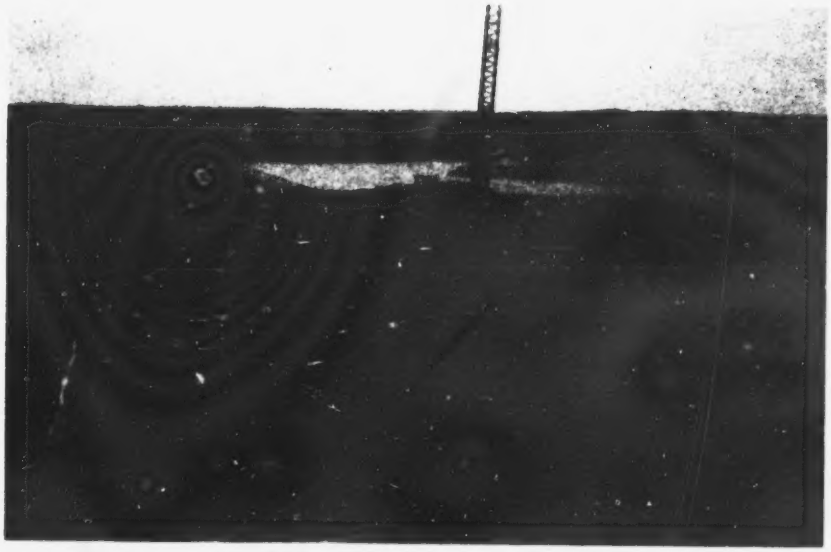


Figure 1-14. - Construction of the cycle of the labyrinth spillway, on March 18, 1983

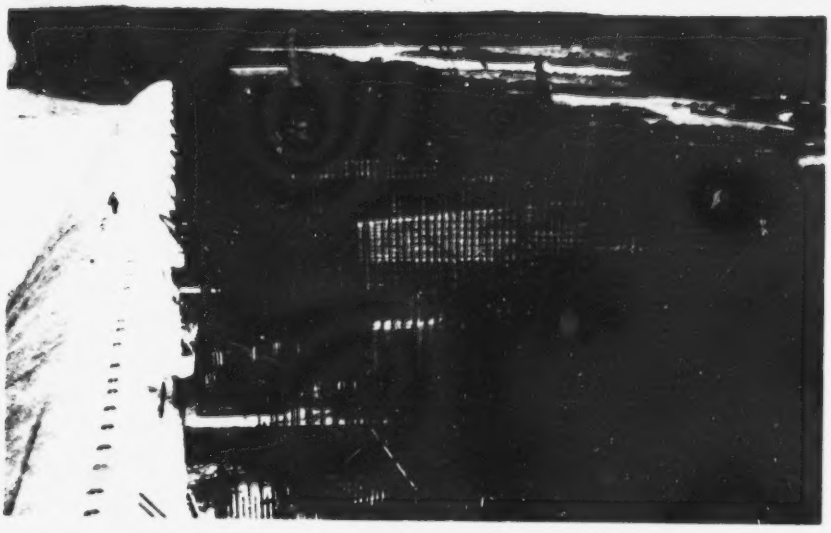


Figure 1-15. - Construction progress of the Ute Dam labyrinth spillway, on August 11, 1983.

Concrete in base slab - over 13,000 yd^3
Cementitious materials - over 7,500 tons
Reinforcing bars - over 7,000,000 lb
PVC waterstops - over 6,000 lin ft
4-inch split drain pipe - nearly 7,000 lin ft

ADDITIONAL MODEL STUDIES AND GENERAL DESIGN GUIDELINES

As a result of the findings associated with the model study of the Ute Dam labyrinth spillway, interest was expressed in better defining the labyrinth parameters. Another site-specific model study was conducted for an auxiliary labyrinth spillway at Hyrum Dam [8]. These studies, combined with some additional testing, produced modified guidelines for labyrinth spillway design. The newly developed design curves will be discussed along with design guidelines and limitations on the structural design.

Hyrum Dam Auxiliary Labyrinth Spillway

The auxiliary labyrinth spillway for Hyrum Dam was designed from the Ute model study data. Hyrum labyrinth was a 12-foot-high, 2-cycle spillway with a design discharge of 9,050 ft^3/s passed with a reservoir head of 5.5 feet (0.5 foot below maximum water surface). The dimensions and parameters of the spillway are:

H_o/P	= 0.5	α	= 8°55'48"
W	= 60 feet	a (U/S)	= 3 feet
w	= 30 feet	a (D/S)	= 1 foot
l/w	= 5	b	= 71 feet
w/P	= 2.5	L	= 300 feet

Principal features investigated during this model study were the spillway approach conditions and the orientation of the spillway [8]. Placing the spillway 19 feet into the reservoir, with curved sidewalls adjacent to the spillway, provided the optimum hydraulic efficiency. Details of the effects of entrance conditions and labyrinth spillway orientation will be discussed in the following sections.

DESIGN CURVES

Hydraulic model results have shown that previously used labyrinth spillway design curves [5] did not provide adequate labyrinth crest length to pass the maximum discharge within the design head value. New sharp-crested labyrinth curves (fig. 1-16) were developed and confirmed by the successful design of both the Ute Dam and Hyrum Dam spillways. However, because these curves were based on sharp-crested weirs, the conversion to the actual prototype crest shape was often tedious and sometimes caused inaccuracies in the design. This led to further model testing and the development of design curves based on a more commonly

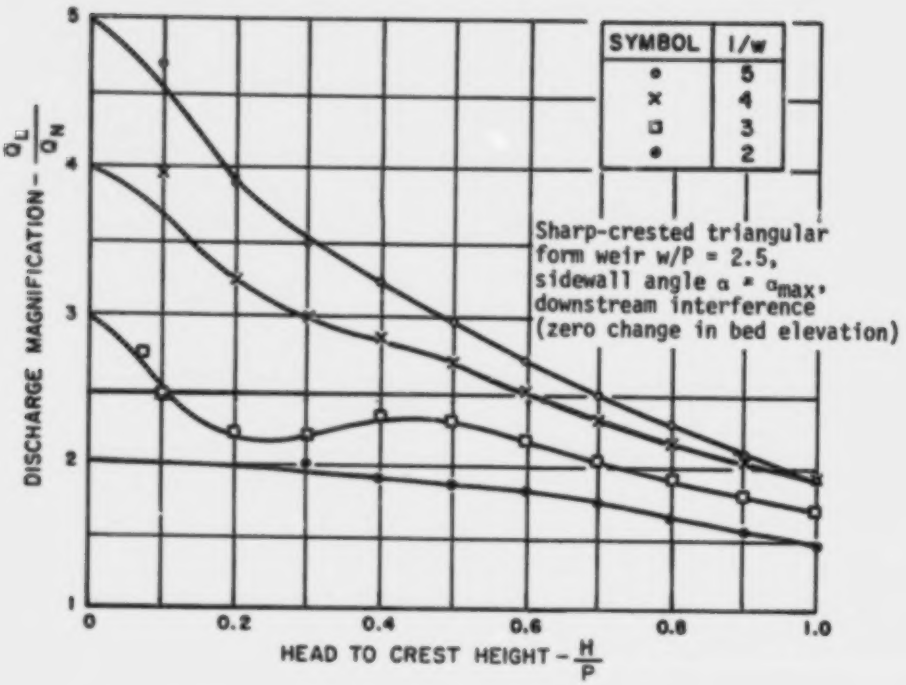


Figure 1-16. - Sharp-crested labyrinth design curves.

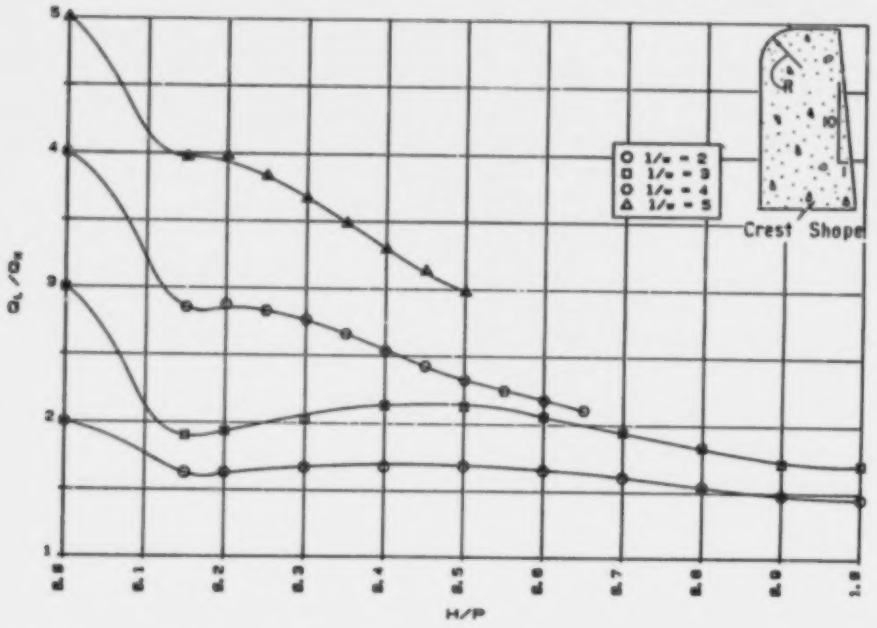


Figure 1-17. - Design curves with quarter-round upstream face, trapezoidal form weir.

shaped crest. Because most previously designed labyrinth spillways have a crest shape with a quarter-round upstream face, which produces a high discharge coefficient, this shape was used in the design curves (fig. 1-17). The design procedure [5] may be simplified by using the design curves based on the quarter-round crest shape, provided the labyrinth spillway under consideration has the same shape.

GENERAL SPILLWAY DESIGN GUIDELINES

The following sections contain general design guidelines for a labyrinth spillway including information on the approach conditions, the spillway placement and orientation, and the performance parameters, w/P, n, and l/w. Also, aeration of the nappe during discharges under low head will be discussed.

Spillway Approach Conditions

The labyrinth geometry makes the spillway sensitive to the reservoir approach flow conditions. The two major factors of the approach condition affecting spillway performance are the direction of the approach flow with respect to the spillway and the shape of the entrance structures immediately upstream of and adjacent to the spillway. Of these, the flow direction is more important because an approach flow parallel to the centerline of the spillway cycles will produce the most uniform flow distribution throughout the spillway and provide a good basis for designing the spillway entrance. The most efficient spillway entrance for most reservoir applications is a curved approach adjacent to each end cycle of the spillway. This will produce uniform approach flow to the end cycles of the spillway. The entrance configuration is very important, particularly when the spillway has only a few cycles, because a significant portion of the total crest length is then affected by the entrance shape.

Spillway Placement and Orientation

The spillway entrance shape should be coordinated with the placement and orientation of the spillway. When installing a labyrinth spillway in a reservoir, the spillway placement is more important than the orientation. Placement should be as far upstream in the reservoir as possible. Such placement will reduce the localized upstream head losses because the area contraction immediately upstream from the spillway is reduced. When the spillway placement has been determined, the orientation or the attachment of the spillway to the abutments or sidewalls should be considered. The importance of spillway orientation is magnified when the reservoir approach conditions are poor or the spillway is placed in a canal. In these cases, with the apexes of the end cycles located upstream, the water surface along the sides is rough, producing a noticeable reduction in head and discharge. The spillway placement, orientation, and entrance are usually determined by the site and availability of a good foundation.

Number of Spillway Cycles and Nappe Interference

The number of spillway cycles should be based on the magnitude of the upstream head, effect of nappe interference, and economics of the design. The number of cycles and spillway height determine the vertical aspect ratio, W/P. In turn, the vertical aspect ratio and the head determine the occurrence of nappe interference. Under high heads the hydraulic efficiency is dependent upon the nappe interference. Nappe interference occurs when the sides of the cycle are close enough that the nappes from the flow over each side intersect or impinge prior to reaching the floor in the downstream channel. This flow condition will reduce the discharge capacity of the spillway. As a general rule, the importance of the vertical aspect ratio and nappe interference increases as the head increases. With normal operating conditions, the vertical aspect ratio should be 2.5 or greater, although this ratio may be lower with low head values because the nappe will be very thin and the spillway will behave almost ideally. An example of the head drop associated with nappe interference is seen on figure 1-18.

Impact Pressures in the Downstream Channels

For the Ute spillway, pressures were measured in the downstream channels parallel to the spillway walls and along the centerline of the spillway cycles. None of the pressures measured were excessive. The pressures were highest parallel to the sidewalls where the jet impinged on the floor after flowing over the crest. However, these pressures and those measured along the cycle centerline decreased as the downstream channel expanded. The pressures will vary according to the tailwater present in the downstream channels, the cycle width, and the geometry of the chute downstream of the spillway.

Labyrinth Spillway Low Flow Conditions

Nappe oscillation and noise will occur when the spillway is operating under low heads. These phenomena are produced by alternating atmospheric and subatmospheric pressures under the nappe. Subatmospheric pressures will increase the flow over the spillway, but should be avoided for structural reasons.

Two methods have been recommended to solve the problem of subatmospheric pressures - placing splitter piers along the spillway side walls and placing crushed stone along the downstream edge of the crest. Splitter piers have been designed for use at Ute and Hyrum Dams. The piers should be located at a distance equal to 8 to 10 percent of the wall length, upstream of the downstream apexes. The height of the piers should vary according to the head range of concern. The piers may be submerged during higher flows. Figure 1-19 shows a spillway cycle passing a low discharge with and without splitter piers. Notice the small splitter piers located on the sides of the left cycle and the break in the nappe in these areas.

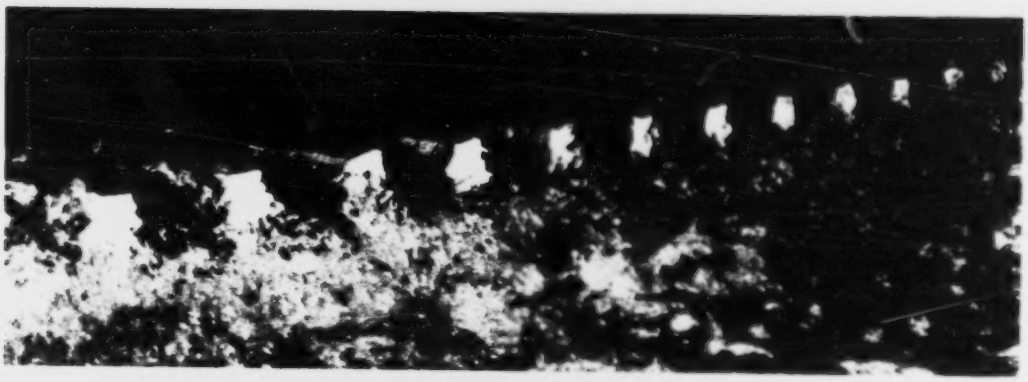


Figure 1-18. - Ute Dam labyrinth spillway with Q = 550,000 ft^3/s, H = 19 feet, and a head loss due to nappe interference.

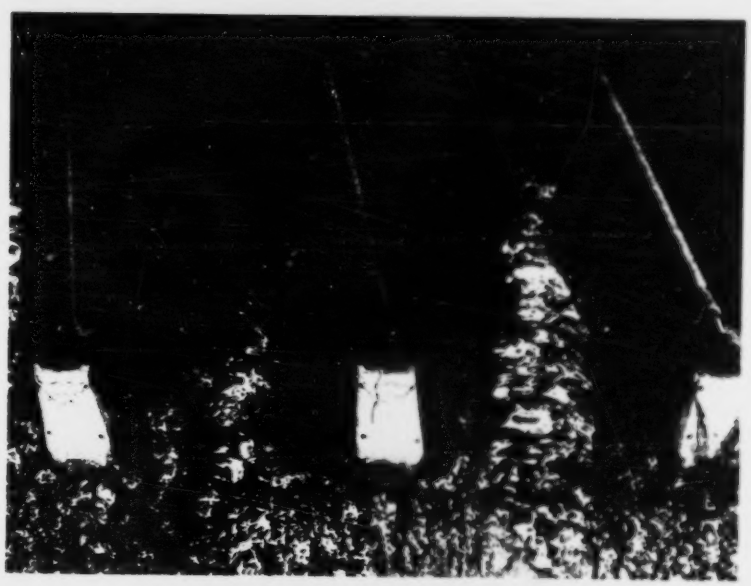


Figure 1-19. - Low flow condition aerated with splitter pier and nonaerated.

Placing crushed stone along the downstream edge of the crest proved successful at Avon Dam [10]. This method, while decreasing the discharge for a given head, successfully provides aeration and is cost effective.

Structural Configuration

Most of the labyrinths built previously are thin, cantilever-type structures because of their ease of construction and hydraulic performance. These labyrinths are relatively short structures with low depths of overtopping. On the other hand, the large labyrinth spillway designed for Ute Dam (H = 19 feet, P = 30 feet) was heavily reinforced to resist the high moments and stresses that could develop under maximum loading conditions. This depth of overtopping and height of wall are near the maximum feasible dimensions because of the difficulty of installing the large amount of reinforcement required. Using a higher labyrinth or a greater depth of overtopping would, most likely, require a gravity section for the walls, reducing the hydraulic efficiency and the economic advantage provided by the labyrinth spillways.

GENERAL APPLICATIONS OF LABYRINTH SPILLWAYS

The labyrinth spillway at Ute Dam was the first labyrinth designed and built by the Bureau of Reclamation, therefore, the spillway required extensive investigation. This included reviewing labyrinths that have been studied and built by other engineering organizations. The location of these spillways and a summary of the major dimensions and discharge characteristics are shown in tables 1-1 and 1-2. Labyrinth spillways have been built with a wide range of sizes and discharge capacities, indicating a variety of potential applications.

The Bureau of Reclamation has considered the use of labyrinths on dams where the discharge capacity of an existing spillway must be increased or where an existing reservoir must be enlarged. Because of their success and the cost savings involved, labyrinth spillways are now being considered for new structures. As the engineering community gains more experience in the design of labyrinths and additional studies are published, the range of applications will increase. Because a labyrinth spillway is suitable almost anywhere an overflow structure is required, labyrinths are an innovative alternative for the design of dams and waterways.

Table 1-1. - Various labyrinth spillways

Name and location	Year built	Total width, ft	Crest length, ft	Design discharge, ft^3/s	Number of cycles
Ute Dam [6], Logan, NM, USBR	1983	840	3,360	550,000	14
Quincy Dam [3], Aurora, CO, CH_2M-Hill	1973	118	348	19,500	4
Mercer Dam [2], Dallas, OR, CH_2M-Hill	1971	18	246	8,000	4
Woronora Dam [4], MWS&DB, Sydney, Australia	1941	484	1,127	36,000	11
Avon Dam [4,10], MWS&DB, Sydney, Australia	1970	448	868	50,000	10
Bartletts Ferry Dam [9] Columbus, GA, Georgia Power Co.	1982	1,230	4,729	240,000	20-1/2
Navet Pumped Storage [11], Trinidad, CO, CH_2M-Hill	1974	180	450	17,000	10
Hyrum Dam [8], Hyrum, UT, USBR	-	60	300	9,050	2
Ohau C. Canal [12], Upper Waitaki Pwr. Dev., Ministry of Works & Dev., New Zealand	-	253	-	19,070	12
Boardman Spillway [1] Boardman Power Project Boardman, OR, Bechtel	-	120	350	13,660	2

Table 1-2. - Cycle data

Project name	Discharge ft	Head ft	Height ft	Width ft	Length ft	a(U/S) ft	a(D/S) ft	b ft	Crest shape
Ute	39,300	19.00	30.00	60.00	241.70	4.31	2.69	114.00	1/4 arc
Quincy	4,875	7.00	13.00	44.50	86.90	2.00	2.00	39.45	1/4 arc
Mercer	2,110	6.00	15.00	18.00	57.90	2.00	1.00	25.94	1/4 arc
Woronora	3,270	4.46	7.25	44.00	102.46	-	-	-	1/4 arc
Avon	5,000	7.10	10.00	44.41	86.80	2.00	2.00	39.40	1/4 arc
Bartletts Ferry	12,000	6.00	11.25	60.00	230.70	1.35	0.19	113.80	1/4 arc
Navet	1,700	5.00	10.00	18.00	45.00	-	-	-	1/2 arc
Hyrum	4,530	5.50	12.00	30.00	150.00	3.00	1.00	71.00	1/4 arc
Ohau C	1,590	3.53	8.20	20.51	123.03	-	-	-	1/4 arc
Boardman	6,830	5.80	9.06	60.00	174.81	0.78	0.38	86.25	1/2 arc

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FUSE PLUG EMBANKMENTS IN AUXILIARY SPILLWAYS DEVELOPING DESIGN GUIDELINES AND PARAMETERS

by

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January 1984

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Denver, Colorado

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LIST OF SYMBOLS

A - Cross sectional area
B - Width of base of fuse plug
b - Width of base of fuse plug downstream from the core
C - Discharge coefficient
C_c - Correlation coefficient
D - Reservoir water depth above fuse plug base
d - Sand grain diameter
d_{50} - 50 percent sand grain diameter
d_{90} - 90 percent sand grain diameter
E - Modulus of elasticity
E_l - Lateral erosion rate
f - Function of
f - Friction factor
F - Froude number
F_c - Critical tractive force
F_w - Weight force
g - Gravitational acceleration
H - Height of fuse plug
h - Pilot channel depth
J - Weir length
k_s - Equivalent sand grain roughness
L - Distance along fuse plug crest
l - Length
M - Structural merit number
p - Top length of pilot channel along crest
P - Pressure
Q - Water discharge
Q_s - Sediment discharge
q - Unit discharge
q_s - Unit sediment discharge
q_s^* - Dimensionless unit sediment discharge
R - Hydraulic radius
R - Reynolds number
R^* - Boundary or grain Reynolds number
S - Water surface slope
S_s - Sand specific gravity
T - Impermeable core thickness
t - Clay core thickness in the model
t' - Time
u^* - Shear velocity
V - Average water velocity
w - Sediment settling velocity
W - Width of fuse plug crest in direction of flow
γ - Specific force of water
γ_s - Specific force of sediment
θ - Angle of core with horizontal
ν - Kinematic viscosity of fluid
ρ - Density of fluid

ρ_s - Density of sediment grain
σ - Surface tension
σ_g - Standard deviation of particle sizes
τ_c - Critical shear stress
τ_o - Bed shear stress
τ^* - Dimensionless shear stress
$\emptyset$ - Friction angle

Subscript $_r$ refers to the ratio between prototype and model values
Subscript $_m$ refers to the model
Subscript $_p$ refers to the prototype

ABSTRACT

The increasing size of design floods is causing dam designers to investigate more cost-effective methods of providing additional spillway capacity. In many cases, auxiliary spillways with fuse plug embankments can provide an economical alternative to passing all the flow through concrete structures.

The Bureau of Reclamation undertook research to help develop design guidelines to be used in cases where a fuse plug embankment would be appropriate. The lateral rates of erosion and discharge coefficients determined during this research can be used to calculate the downstream flow with a computer flood-routing study.

Fuse plug embankment models were used during the research to study the erosion process. The models were designed according to the Froude law with adjustments made to compensate for low Reynolds numbers affecting the sediment transport rate.

INTRODUCTION

Dam safety concerns have renewed interest in the use of fuse plugs as an economical alternative to installing gates in auxiliary spillways. A fuse plug is an embankment designed to wash out in a predictable and controlled manner when flow capacity in excess of the normal capacity of the service spillway and/or outlet works is needed. An uncontrolled overflow spillway with a high crest elevation would serve the same purpose; however, the uncontrolled spillway would have to be very wide to pass a large flow. A fuse plug controlled auxiliary spillway has the advantage of developing deeper flow after the embankment is washed away, thus allowing the spillway channel to be narrower. The increasing size of design floods for both existing and new dams is a major problem facing dam designers. Several methods are now available to provide spillway capacity for infrequently occurring floods. One of the most common and least expensive alternatives is to provide flood surcharge. Additional surcharge space, however, may not be available for several reasons: development along the reservoir shoreline, raising the dam crest may be impracticable, or additional reservoir head could result in damaging flows to existing waterway structures. Another alternative is to provide an auxiliary spillway. Constructing a fuse plug across the auxiliary spillway would restrict the operation of that waterway to floods of infrequent occurrence, much the same as the installation of gates. In many cases, auxiliary spillways with fuse plug embankments can provide an economical alternative to passing all of the flow through concrete structures. The fuse plug embankment is designed to preclude use of the auxiliary spillway during minor floods.

The auxiliary spillway may be located on an abutment of the dam or at some location on the reservoir rim, provided the discharge can be safely directed into an existing water course. The additional discharge capacity provided by an auxiliary spillway may be required for existing reservoirs when hydrometeorological data acquired since construction have resulted in a revised IDF (inflow design flood) that is too large for the available surcharge space and the existing waterway structures to control safely. In the case of new construction, an auxiliary spillway, in combination with conventional structures, may provide a favorable, economical alternative for making required reservoir releases.

CONCLUSIONS

- A properly designed fuse plug embankment will wash out in an orderly and predictable manner when additional flow capacity is needed to pass a large flood through a reservoir. The embankment will preclude use of the auxiliary spillway during small floods.

- The rate of washout (lateral erosion rate) is a function of the erosion rate of the embankment material and not a function of the impermeable core strength.

- The erosion rates and discharge coefficients determined in this study can be used in flood-routing computer programs to help design fuse plug embankments.

- The sand filter, embankment material, and gradation have significant effects on erosion rates.

- A model design method is described that compensates for the fact that the Reynolds number is normally too low to properly simulate sediment transport in a Froude scale hydraulic model. This method uses settling velocity adjustments and dimensionless unit sediment discharges to adjust the model grain sizes and/or the model sediment density.

DESIGN GUIDELINES

A fuse plug is a zoned earth and rockfill embankment designed to wash out in a predictable manner when breached. The embankment materials are selected and placed so that a constant rate of lateral erosion will occur as the structure is breached.

The fuse plug is designed as a dam, stable for all conditions of a reservoir operation except for a flood that will cause it to breach. The washout of a fuse plug should begin at a preselected location; a washout caused by general overtopping of the entire fuse plug should not occur.

The preferred method is to initiate breaching of the fuse plug with reservoir water. When the reservoir level reaches a predetermined elevation, a low spot in the embankment crest, called a pilot channel, will be overtopped. By placing highly erodible materials in the pilot channel section, breaching will occur rapidly, and the remainder of the fuse plug embankment will wash out laterally at a constant, predictable rate without overtopping. The auxiliary spillway flow will increase at a constant rate. This automatic breach feature is desirable because it reduces the possibility of mechanical or human error at a time when operation of a flood relief mechanism is critical. When a wide auxiliary spillway is required, it may be desirable to sectionalize the fuse plug with concrete separation walls. By using successively higher elevations for the fuse plug crests and pilot channels in each section, the washout process can be matched with successively less frequently occurring floods. The entire fuse plug would not wash out unless the entire capacity of the auxiliary spillway was needed.

The rate of lateral erosion as the fuse plug washes out is of primary importance. The rate of increase in downstream flow is not only dependent upon the rate of lateral erosion, but also upon the elevation of the reservoir. The rate of lateral erosion depends on the gradations and types of materials used to construct the fuse plug, the depth of flow above the base of the fuse plug, and the geometry of the fuse plug section (crest width, angle of the outer slopes, and configuration of the zoning).

The total discharge through an auxiliary spillway with a fuse plug embankment is controlled by the elevation of the grade sill or nonerodible foundation beneath the fuse plug, the width of the spillway channel, and the depth of water above the grade sill or foundation. The maximum depth of water above the base of the fuse plug can be controlled by choosing a pilot channel elevation, which will be breached at a selected time during the IDF. It may be desirable, for instance, to breach the fuse plug early during a flood with

a high peak. This timing would provide double use of some of the surcharge storage space by evacuating that space early in the flood.

As a general rule, fuse plugs should be designed to operate only for floods with recurrence intervals that are long relative to the economic life of the project. Fuse plugs should not be designed to operate for floods with recurrence intervals less than 100 years.

The rates of lateral erosion and coefficients of discharge obtained from curves resulting from Bureau of Reclamation research can be used in a computer flood-routing program to predict the downstream flows and maximum reservoir water surface elevation. The effects of varying the rate of lateral erosion and the elevation of the pilot channel on the reservoir elevation and auxiliary spillway discharge can be determined.

The auxiliary spillway should be designed according to standard practice. The flow velocities must be sufficient for the water to carry the eroded fuse plug material downstream to avoid clogging of the return channel to the natural stream course. If excessive sediment deposition is anticipated, a site-specific model study may be required to design the return channel.

LITERATURE ON FUSE PLUG EROSION

In the past, fuse plug embankments have been designed and constructed for mine tailing dams, on levees, and to control flow in auxiliary spillways. However, there has not been a documented case of a fuse plug controlled spillway actually operating. Most of the information in the literature is associated with studies conducted in 1959 to design a fuse plug controlled spillway for the Oxbow Project, on the Snake River between Idaho and Oregon [1, 2, 3]*.

The Oxbow Project has two spillways, each with a design flow capacity of 4,250 m^3/s (150,000 ft^3/s), which is on the order of the 100-year flood. The total discharge capacity of 8,500 m^3/s (300,000 ft^3/s) corresponds to the inflow design flood. The original design required three radial gates to control each spillway. Later, the Idaho spillway, on the right abutment, was changed to fuse plug control [3]. The studies conducted to confirm the design assumptions included 1:20 and 1:40 scale model tests in the laboratory and a 1:2 scale field test at the damsite.

Another study of erosion mechanics and washout time rates of erodible control embankments was made using hydraulic models at the University of Windsor, in 1977 [4]. This study analyzed theoretical equations and compared calculations with model results.

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* Numbers in brackets refer to entries in the Bibliography.

THE MODEL

Description

The model was designed to simulate typical prototype fuse plugs from 3 to 9 m (10 to 30 ft) high. The model embankments were from 0.15 to 0.38 m (0.5 to 1.25 ft) high and 2.7 m (8.8 ft) long at scales of 1:25 and 1:10, respectively (fig. 2-1). The model size was based on the maximum flow available in the laboratory. A flow of 0.61 m^3 (21.5 ft^3/s) was made possible by using two pumps operated in parallel.

The overall model was 14 m (46 ft) long by 8 m (26 ft) wide. Flow entered the model through two 0.3-m (1-ft) pipes and passed through a rock baffle into a 7.6-m (25-ft) by 5.2-m (17-ft) by 1.5-m (5-ft) deep headbox (fig. 2-2). The headbox simulated a reservoir in a prototype structure. The water surface level in the headbox was controlled by a 7.6-m (25-ft) long adjustable-height weir along one side. Water flowing over the adjustable weir plunged into a side channel and then passed through a flow measurement weir. The flow measurement weir was calibrated for flow versus water surface elevation in the side channel. The measurement weir was a combination type. The lower 0.31 m (1 ft) of weir was a 90° V-notch. Above the V-notch were 0.61-m (2-ft) extensions on each side at a 15° angle with horizontal. Above these 15° extensions were 0.46-m (1.5-ft) long vertical sides. The calibration curve for the measurement weir is shown on figure 2-3. The calibration was done in three parts corresponding to the three different sections of the weir. During each test, the calibration curve shown on figure 2-3 was used to compute the discharge through the measurement weir.

The fuse plug embankment platform was located at the end of the headbox. The platform was placed at an elevation 0.76 m (2.5 ft) above the headbox floor. One end wall of the platform was constructed from transparent plastic to observe the initial breach and lateral erosion process (fig. 2-4). A sloping platform, downstream from the horizontal fuse plug platform, led to a tailbox where the sediment was deposited before the water returned to the laboratory supply reservoir. A typical test used the following procedure:

1. With the adjustable control weir at a low level, the valves controlling the two inlet pipes were opened; the entire flow of 0.61 m^3/s (21.5 ft^3/s) entered the headbox, passed over the control weir and through the measurement weir.

2. The test was started by raising the reservoir water surface (with the control weir) to a predetermined level where water began flowing through the pilot channel.

3. As the fuse plug embankment washed away, more water passed through the breach. The water surface was kept at a constant level by gradually raising the control weir.

4. The flow through the measurement weir, the level of the water surface in the reservoir, and the time were recorded continuously. Each test was videotaped; still photographs and slides were also taken.

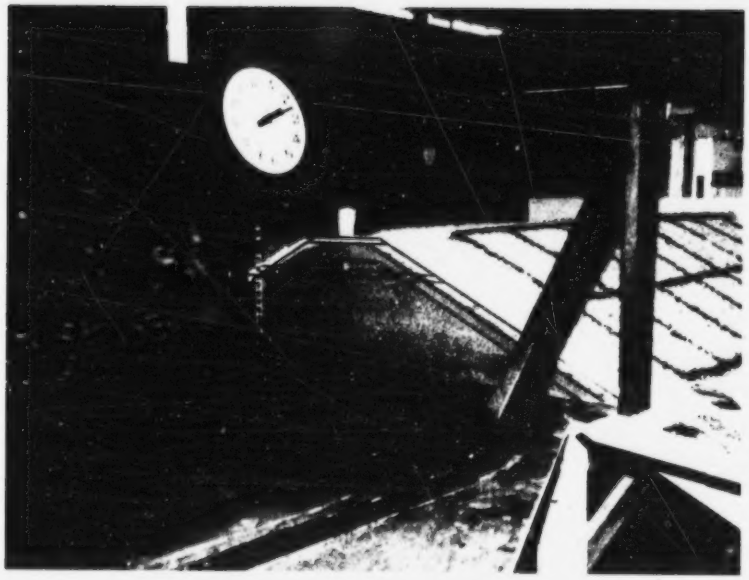


Figure 2-1. - Model fuse plug embankment, test No. 7.

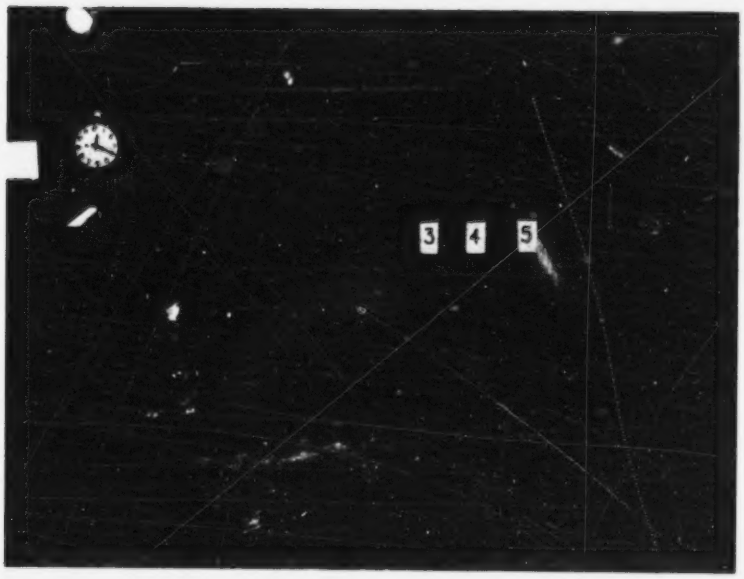


Figure 2-2. - Model operation, test No. 4. Water enters the model through the two pipes in the background. The water surface level is controlled by the long weir along the side.

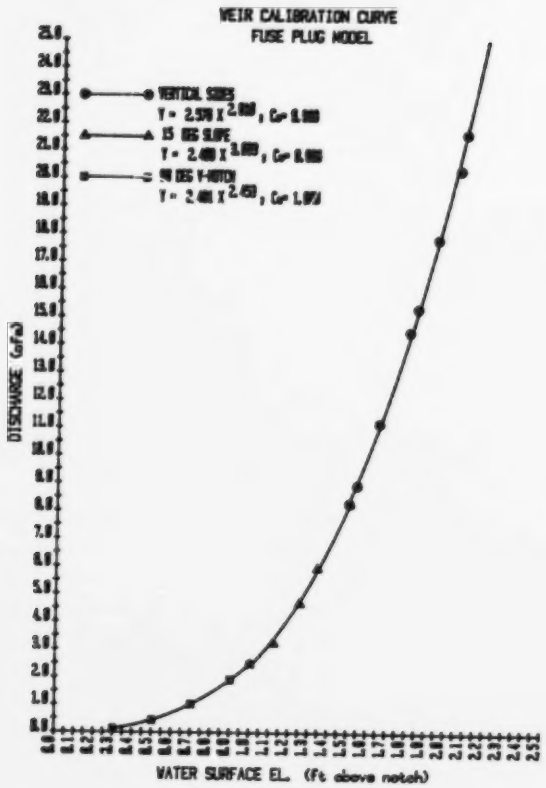


Figure 2-3. - Flow measurement weir calibration curve.

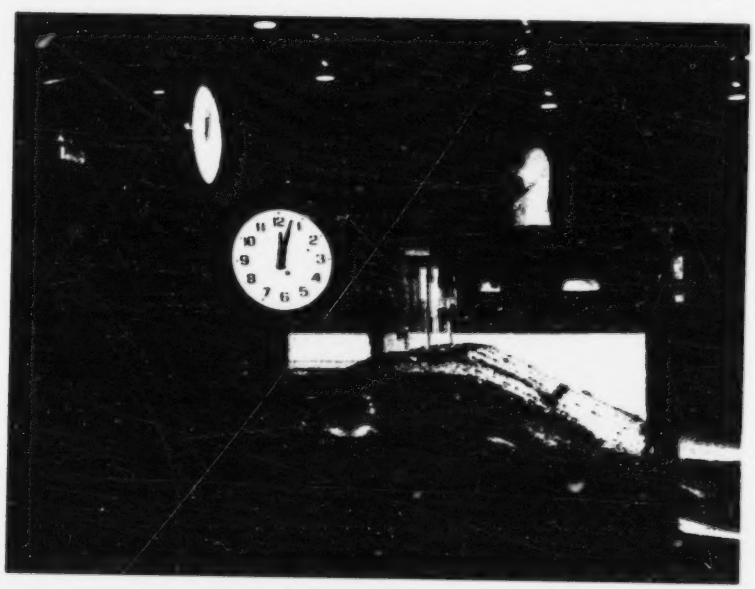


Figure 2-4. - Initial breach viewed through end wall, test No. 8.

5. Flows through the breach were computed by subtracting the measurement weir readings from the total (initial) flow.

Similitude

Hydraulic model studies were used because of the large number of variables involved in hydraulic flow problems, together with specific boundary configurations. The physical behavior of a model simulates (in a known manner) the physical behavior of the prototype.

There are several types of similarity. Geometric similarity exists when the ratios of all homologous dimensions, between the model and the prototype, are the same. The geometric scale ratio, or length ratio, is denoted by $L_r = L_p/L_m$, where the subscripts $_p$ and $_m$ refer to prototype and model, respectively.

Kinematic similarity, or similarity of motion, implies that the ratios of velocities and accelerations between the model and prototype are equal.

Dynamic similarity requires that the ratios of homologous forces between the model and prototype be the same. In hydraulic problems, the primary forces that influence the flow are the forces due to gravity, viscosity, pressure, surface tension, and elasticity. The inertial force is the vector sum of all the others [5]. The following dimensionless numbers relate inertial force to each of the forces listed above.

Froude number (inertia/gravity), $F = \frac{V}{\sqrt{Lg}}$ (1)

Reynolds number (inertia/viscosity), $R = \frac{VL}{\nu}$ (2)

Euler number (inertia/pressure) $= \frac{\rho V^2}{\Delta p}$ (3)

Weber number (inertia/surface tension) $= \frac{\rho L V^2}{\sigma}$ (4)

Cauchy number (inertia/elasticity) $= \frac{\rho V^2}{E}$ (5)

It is apparent that a model fluid cannot simulate all of these properties at once. However, in most cases several of the forces will be absent or negligible in the model. Therefore, a model can usually simulate the critical forces in the prototype for a certain type of flow.

Hydraulic similitude. - The flow through a fuse plug is primarily determined by gravity and inertia forces; the other forces may be neglected. The ratios between the model and prototype are determined from the Froude law (equation 1). The scale relations according to the Froude law are listed below:

Ratio		Scale relation (prototype/model)
Length	=	L_r (geometric ratio)
Area	=	L_r^2
Volume	=	L_r^3
Time	=	$t_r' = \frac{L_r}{g_r}^{1/2} = L_r^{1/2}$ (for $g_r = 1$)
Velocity	=	$V_r = L_r/t_r' = L_r^{1/2}$
Discharge	=	$Q = L_r^{5/2}$

Dimensional analysis. - Vanoni [6] discusses the important variables involved in the present knowledge of sedimentation in a section on "Fundamentals of Sediment Transport." He reduced the sediment discharge rate, Q_s, to the following relationship. These symbols are defined in the list at the front of this paper.

$$Q_s = f(Q, d, \nu, \rho, \rho_s, \sigma_g, w, g) \qquad (6)$$

The following sections will address each of the variables in equation 6.

Sediment transport. - Models involving erosion of noncohesive bed material must simulate tractive stress (τ_o), since the tractive stress causes the drag force required to overcome the forces holding a particle in place (fig. 2-5).

The tractive stress on a particle fluctuates because of the turbulence. The drag force and turbulence are a function of the Reynolds number (equation 2). Therefore, a model operated according to Froude scaling does not necessarily simulate the tractive forces and sediment erosion accurately. In some models the sediment sizes must be adjusted to compensate for a Reynolds number that is too low.

Shields developed a diagram relating dimesionless shear stress (τ^*) to a boundary or grain Reynolds number (R^*). Shields used this diagram to define critical shear stress, τ_c (the stress required for incipient motion of sediment). This concept has been expanded by others to include dimensionless unit sediment discharge.

$$(q_s^* = \frac{q_s}{u^* d})$$

Taylor [6] has shown that dimensionless unit sediment discharge at low transport levels falls very close to the Shields curve for incipient motion. Figure 2-6 shows that for R^* less than 100, the sediment discharge rate increases for a given dimensionless shear (τ^*). The dimensionless shear is actually a function of the Froude number and the density ratio of the sediment and the fluid.

$$\tau^* = \frac{\tau_o}{(\gamma_s - \gamma)\, d} = \left(\frac{u^{*2}}{gd}\right)\left(\frac{\gamma}{\gamma_s - \gamma}\right) \qquad (7)$$

where:

$$R^* = u^* d/\nu$$

$$u^* = \sqrt{\tau_o/\rho} = \sqrt{gRS} = V\sqrt{f/8}$$

$$\frac{u^{*2}}{gd} = \text{Froude number}$$

and,

$$\left(\frac{\gamma}{\gamma_s - \gamma}\right) = \text{density ratio}$$

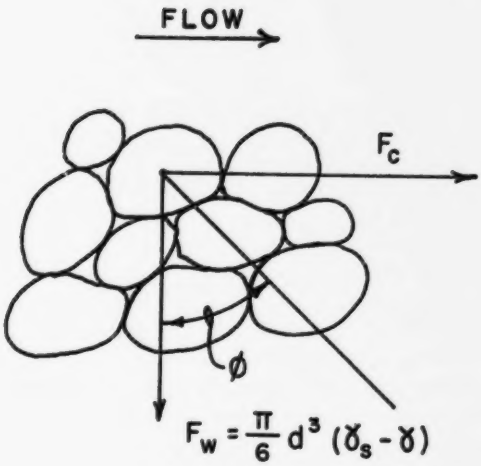


Figure 2-5. - Forces on a sediment particle.

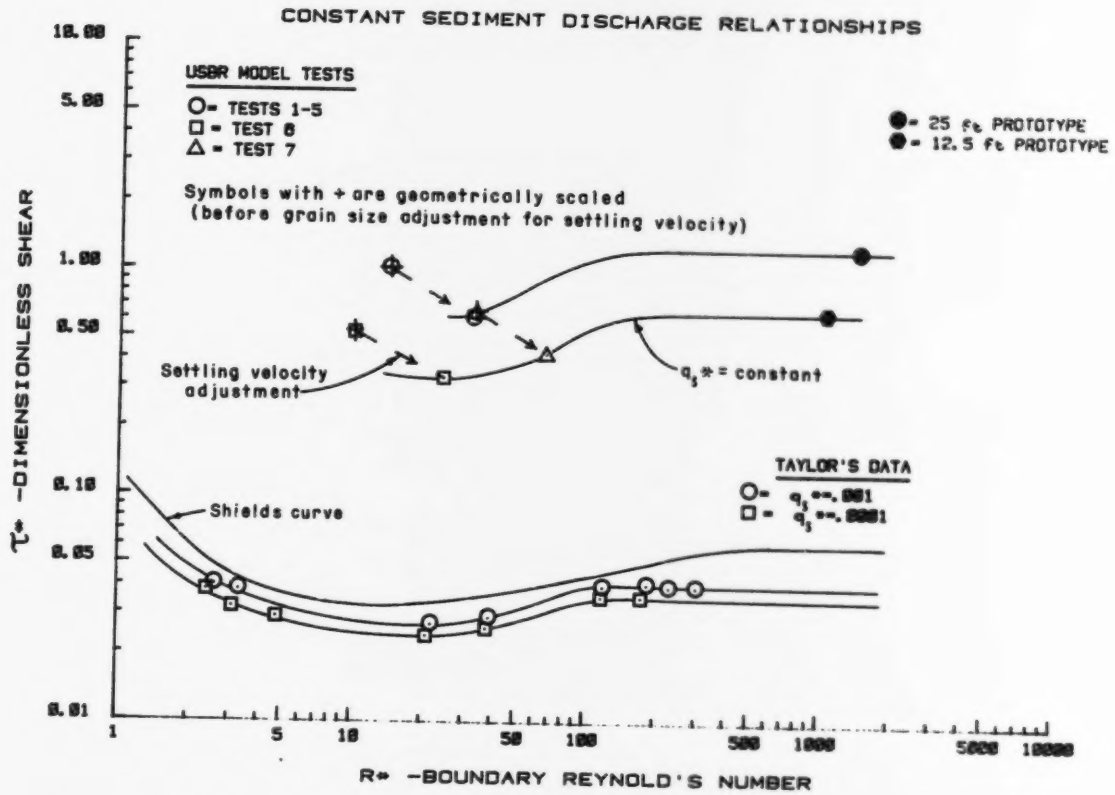


Figure 2-6. - Dimensionless unit sediment discharge versus dimensionless shear and boundary Reynolds number.

Figure 2-6 shows that a model scaled according to the Froude law (τ^* = constant) will erode too rapidly in the range from $5 < R^* < 100$. A diagram of settling velocity (fig. 2-7) also illustrates that small particles settle at a slower rate and require less tractive force to move.

For particle diameters above 1 mm, the settling velocity (w), is a function of the particle diameter (d) to the 1/2 power. This is consistent with Froude scaling ($V_r = L_r^{1/2}$).

Settling velocity adjustment. - By increasing the size of a model sediment grain, the settling velocity can be corrected to Froude scaling. According to geometric scaling, a 1:10 scale model of prototype sand 2.0 mm in diameter would use sand 0.2 mm in diameter. However, the settling velocity would then be about 0.02 m/s (see fig. 2-7) when it should be 0.049 m/s, according to Froude scaling. If the model particle size is increased from 0.2 mm to 0.4 mm, the settling velocity is corrected to 0.049 m/s, the proper value for Froude scaling.

The effect of the settling velocity adjustment on the dimensionless sediment discharge rate (q_s^*) is shown on figure 2-6. The symbols with a "+" are computed according to geometric scaling. Note that the model values of τ^* are about the same as the prototype values they simulate. Tests 1-5 simulate the 25-foot prototype, and tests 6 and 7 simulate the 12.5-foot prototype. However, the model value of q_s^* must be the same in the model and prototype to properly scale the time rate of sediment transport. When the model grain sizes are adjusted for settling velocity (as described above) the value of τ^* decreases while the value of R^* increases. This brings the model value of q_s^* much closer to the projected prototype value of q_s^* (see fig. 2-6). In this study, the model grain sizes were computed using this method of settling velocity adjustment to account for the low boundary Reynolds number. This method applies to noncohesive materials in the model and the prototype, and must be checked for each grain size and each model flow condition. If the model Reynolds number (R^*) was even lower, the model sediment density (ρ_s) could be adjusted to match q_s^*.

Erosion rate scale ratio. - After the settling velocity adjustment described above, the erosion rate scales according to the Froude law. Velocities scale according to $L_r^{1/2}$; therefore, the erosion rate (E_1) also scales according to $L_r^{1/2}$.

The model tests conducted for the Oxbow Project used prototype materials in the model. For these materials, Tinney and Hsu [1] concluded that the erosion rate ratio would be, $(E_1)_r = L_r^{1/3}$. Chee [4] derived the following equation for the erosion rate ratio,

$$(E_1)_r = L_r^{(0.375)} (S_s - 1)_r^2 \, d_r^{0.13}$$

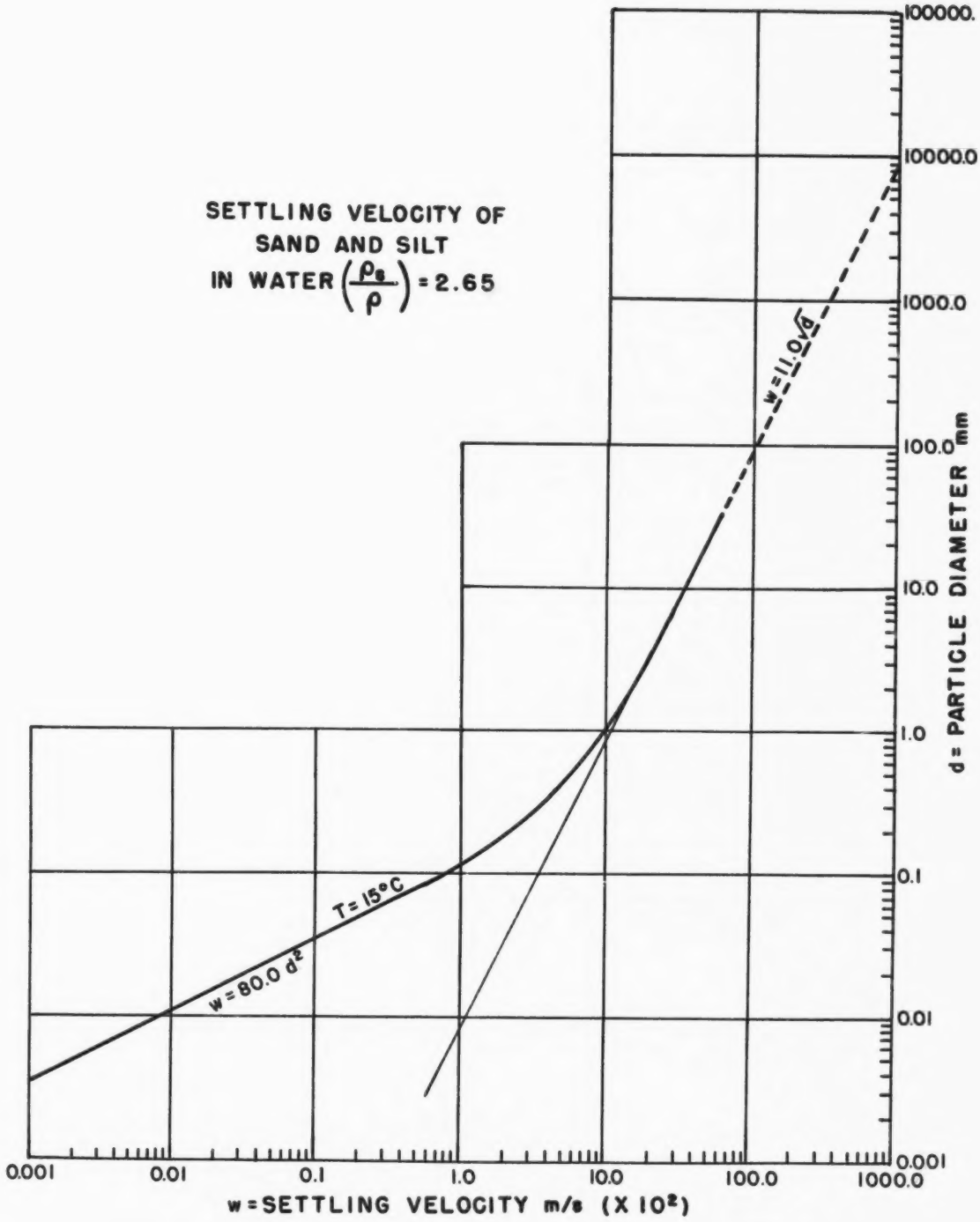


Figure 2-7. - Settling velocity of sand and silt in water $(\rho_s/\rho) = 2.65$.

where S_s is the sand specific gravity. If $(S_s-1)_r = 1$ and $d_r = L_r$ (geometric scaling), then $[E_l]_r = L_r^{0.505}$. This is very close to the Froude scaling ratio obtained in the analysis, $(E_l)_r = L_r^{1/2}$. For $(S_s-1)_r = 1$ and $d_r = 1$, the erosion rate ratio derived by Chee is also close to that derived by Tinney and Hsu, $(L_r)^{.375}$ versus $(L_r)^{1/3}$.

However, it is much more desirable to use model particles of a size close to the geometric scale, because prototype-sized materials in a model do not simulate the correct number of particles per unit volume or the contact force between the grains. In addition, the drag force on the model particles is not correct when the model particles are prototype size.

Embankment design. - The model fuse plug embankment was designed with the same zones found in most zoned earth or rockfill dams. The arrangement of these zones is shown on figure 2-8. The main difference between a fuse plug embankment and a typical rockfill or earthfill dam is the arrangement of the impervious core. The core is inclined so that when the downstream material is washed away, pieces of the core break off because of bending under its own weight and the water load. The core material is normally silt or clay. The sand filter will prevent piping through cracks that may develop in the core. The filter will also keep windblown silt and clay from infiltrating the downstream embankment material. The compacted sand and gravel and compacted rockfill are designed to be noncohesive and easily erodible once the washout process begins. The prototype gradation curves for each zone are shown on figure 2-9. A range of acceptable sizes are shown with the gradation simulated in the model study indicated by a dashed line.

The model and prototype gradation curves for each zone are shown on figures 2-10 through 2-13. These model gradation curves were determined by making settling velocity adjustments to the grain sizes determined by geometric scaling.

The pilot channel section was designed to wash out quickly once the water has started to flow through the pilot channel. A slightly larger rockfill material with fewer sand sizes was used in this section to ensure a rapid break. The prototype pilot channel was designed to be 0.9 m (3 ft) deep, allowing for a 0.3 m (1 ft) water depth and 0.6 m (2 ft) of freeboard. This depth of water was determined to be adequate to initiate a breach during the Oxbow study. The width of the pilot channel was investigated in this model study. The side slopes of the pilot channel were set at 1:1; however, this value could be varied in the prototype. The gradation and compaction of the noncohesive materials are very important factors in determining the erosion rate. As the materials are compacted, more tractive force is required to remove the grains since there are more grains per unit volume and more contact and interlocking between the grains. A well-graded mixture of grains will require more tractive force to erode than uniform-sized material. The smaller particles will fill the voids between the larger particles, making the mixture heavier per unit volume and creating more contact between the particles.

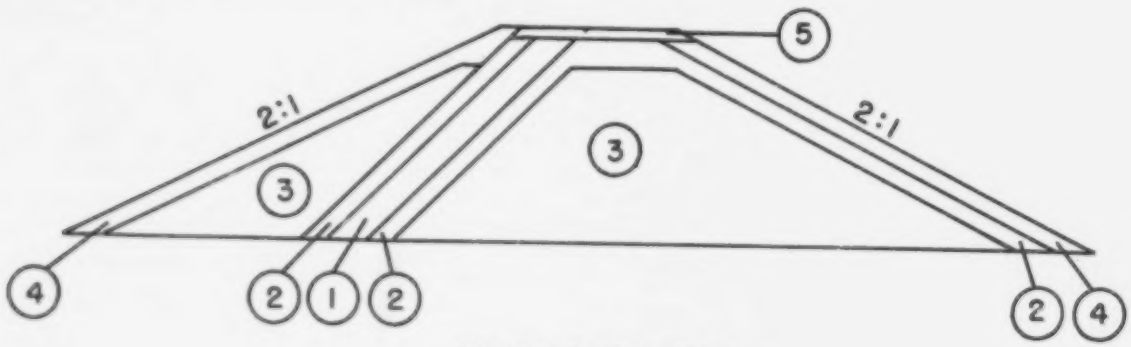
(1) CORE MATERIAL

(2) SAND FILTER

(3) COMPACTED SAND AND GRAVEL

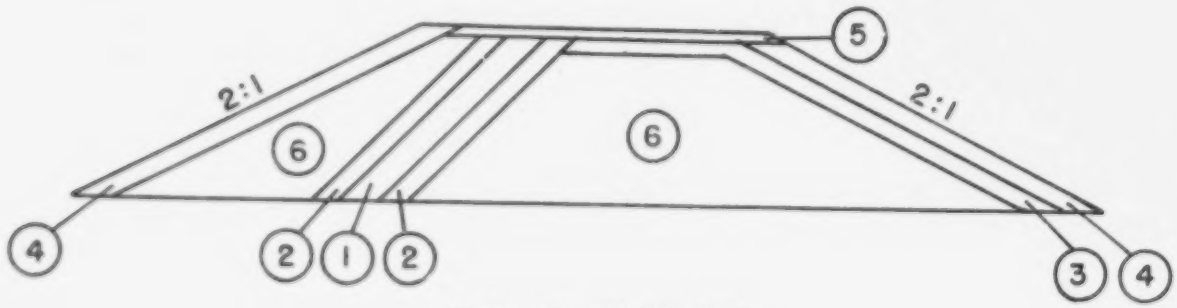
(4) SLOPE PROTECTION

(5) GRAVEL SURFACING



EMBANKMENT

(6) COMPACTED ROCKFILL



PILOT CHANNEL

Figure 2-8. - Fuse plug embankment and pilot channel.

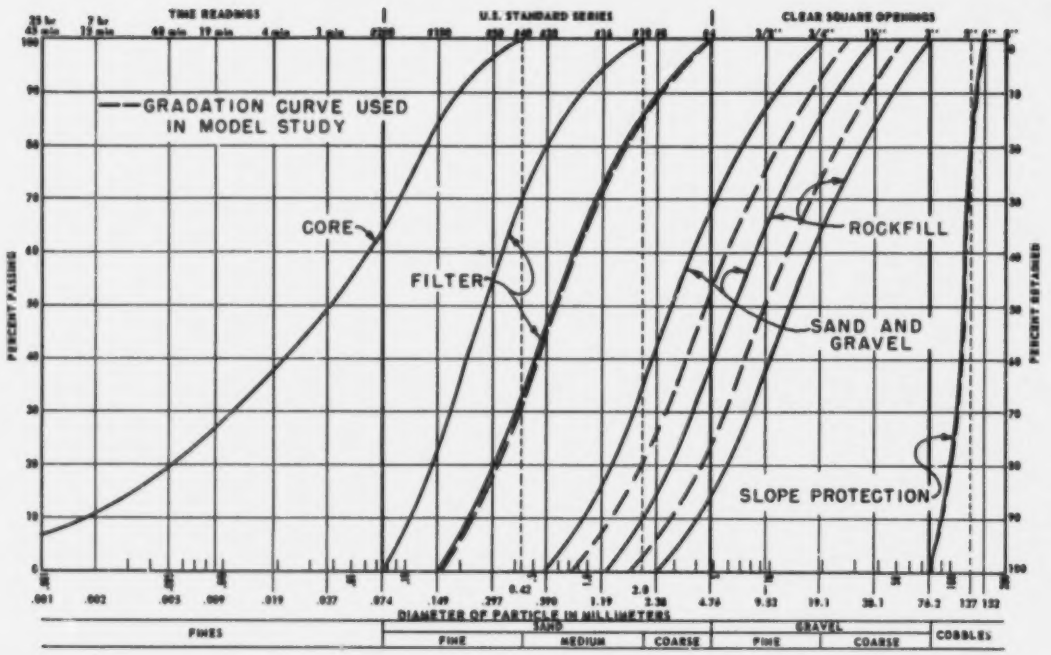


Figure 2-9. - Prototype gradation curves.

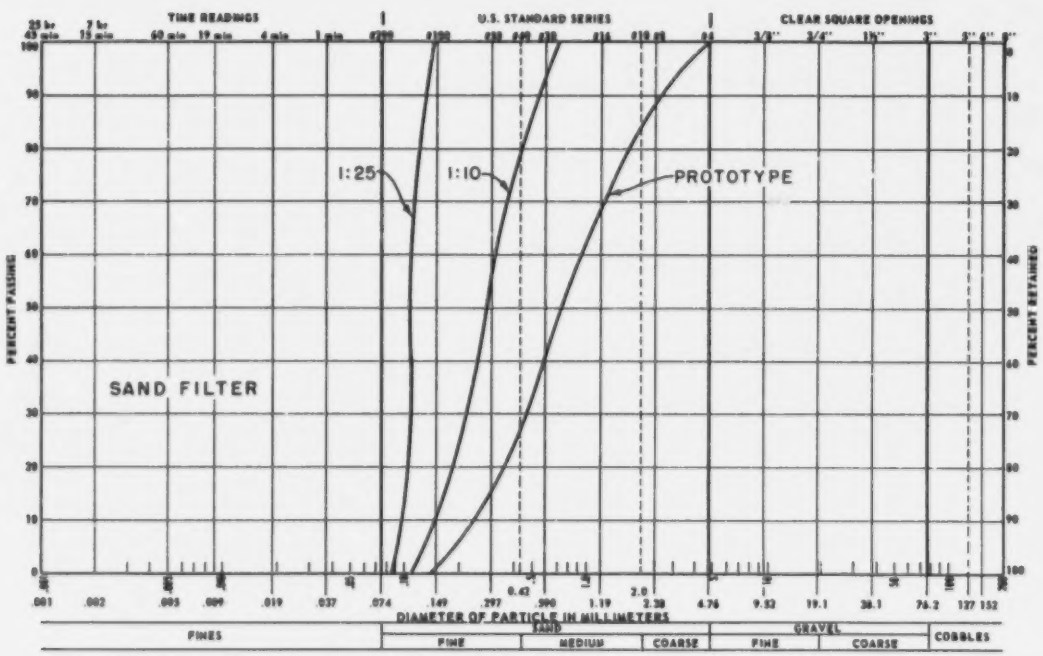


Figure 2-10. - Gradation curves, sand filter.

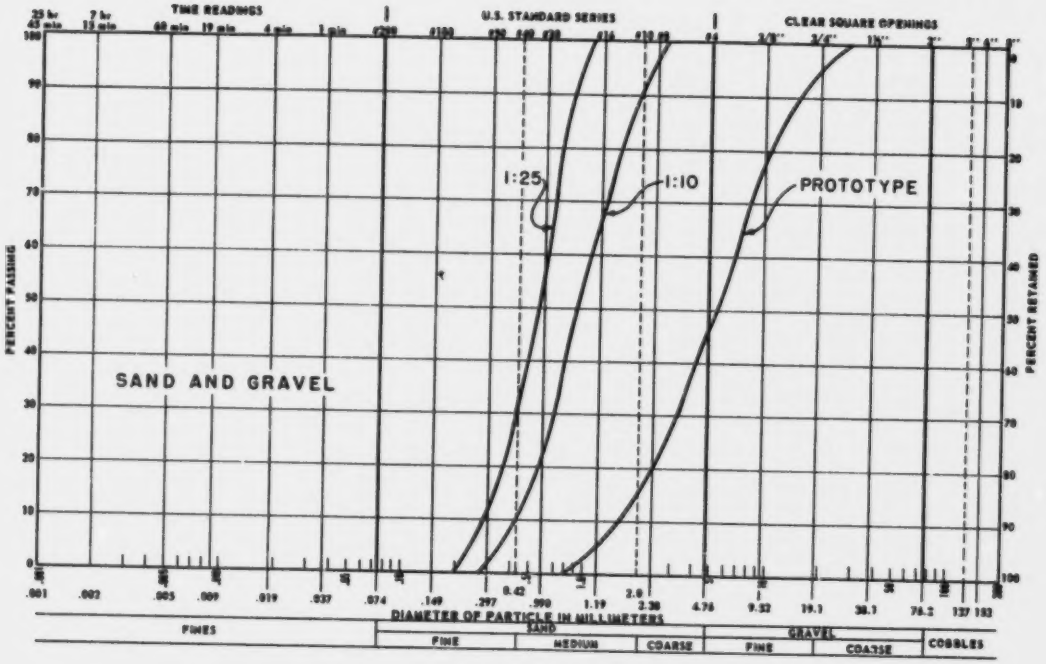


Figure 2-11. - Gradation curves, sand and gravel.

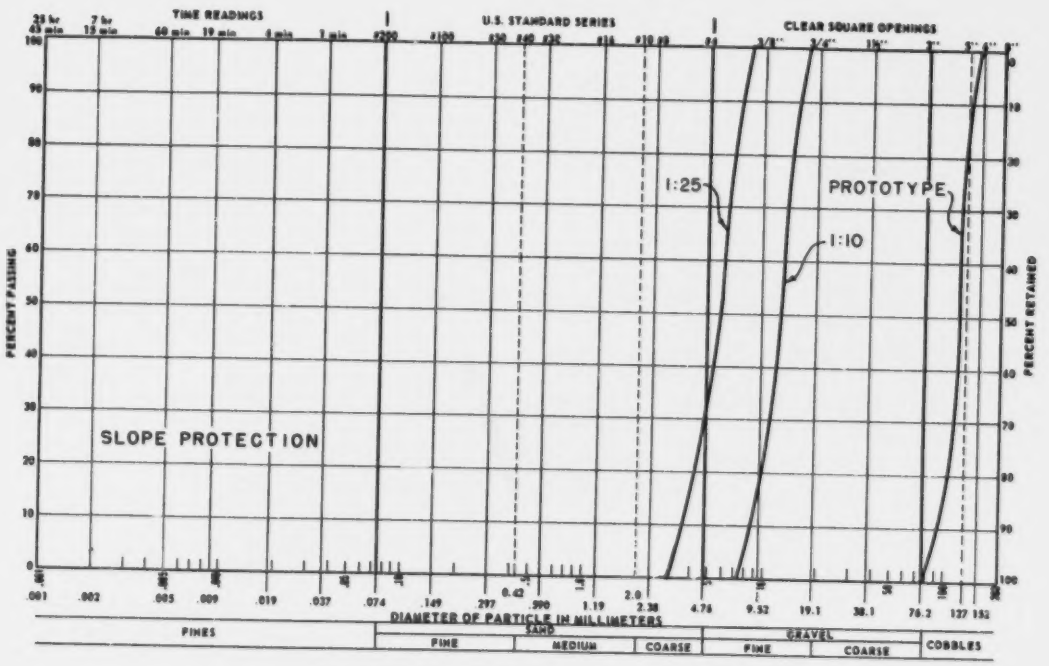


Figure 2-12. - Gradation curves, slope protection.

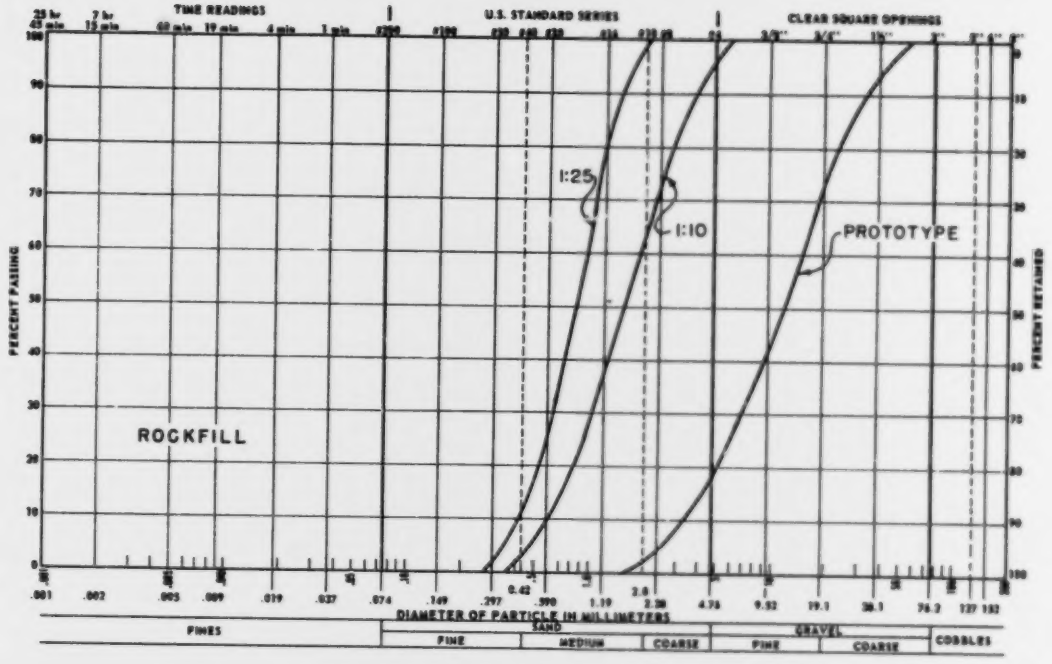


Figure 2-13. - Gradation curves, rockfill.

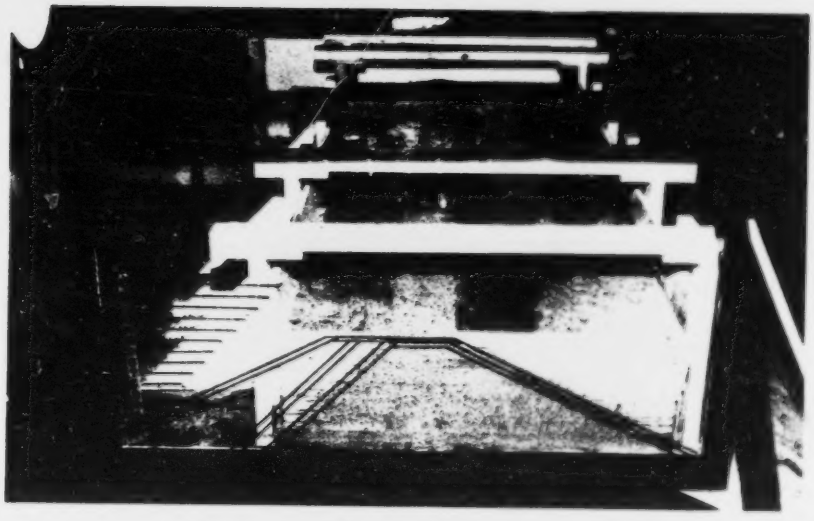


Figure 2-14. - Model placement.

For these reasons, great care was taken in placing the model fuse plug embankments. Relative density tests were conducted on the model materials (see gradation curves on figs. 2-10, 2-11, and 2-13.) The materials were then placed in the model at 70 percent relative density.

The method used consisted of placing a predetermined weight of material in a given volume to obtain 70 percent relative density. The proper proportion of each of the material sizes was mixed to obtain the desired gradation. A known weight of the mixture was then placed in a known volume in the model (fig. 2-14). The volume in the model was controlled by using wooden forms 38 to 51 mm (1-1/2 to 2 in) thick. The main embankment downstream from the core (zones 3 and 6 on fig. 2-8) was placed in layers using this method. The compaction was obtained by using a compressed-air powered vibrator and tamping tools. When the placement of this zone was complete the forms were removed and the corners trimmed to the correct slope. The sand filter, core, embankment upstream from the core, slope protection, and gravel surfacing were then weighed and applied using templates and tamping tools. A completed model fuse plug embankment is shown on figure 2-15.

The model fuse plug embankments were all placed in the same manner to ensure consistency between tests. Since the materials were similar to the prototype in gradation (σg) and relative density, the erosion rates determined in the model should correctly simulate prototype erosion rates.

Structural similitude. - The impervious core was not simulated as part of the hydraulic modeling because the cohesive clay portion does not fail as a result of sediment erosion. The core is designed to break off in pieces from the weight of the water and embankment material above it, as the noncohesive material downstream washes away. Figure 2-16 is a schematic diagram illustrating the failure mode of the materials in the pilot channel. The core fails in a similar manner during the lateral erosion process, as the material on the face of the embankment downstream from the core is washed away.

The structural behavior of the core material was simulated qualitatively, because the prototype core material strength will vary a great deal. A structural analysis of the prototype core as a cantilevered slab indicates that only about 0.88 m (2.9 feet) of core would overhang horizontally before it would break (assuming a high tensile strength in the core of 6895 KPa (1000 lb/in^2)).

The structural behavior of the core material is governed by gravity and elasticity forces. The structural merit number (M) is the dimensionless ratio of gravity forces to elasticity forces.

$$M = \left(\frac{\gamma L}{E}\right) \qquad (8)$$

where E = Modulus of elasticity of the core.

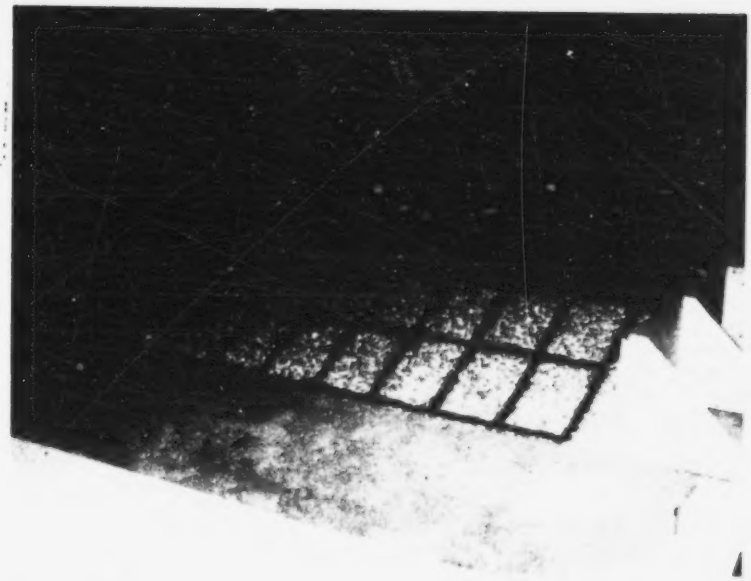


Figure 2-15. - Fuse plug embankment after placement.

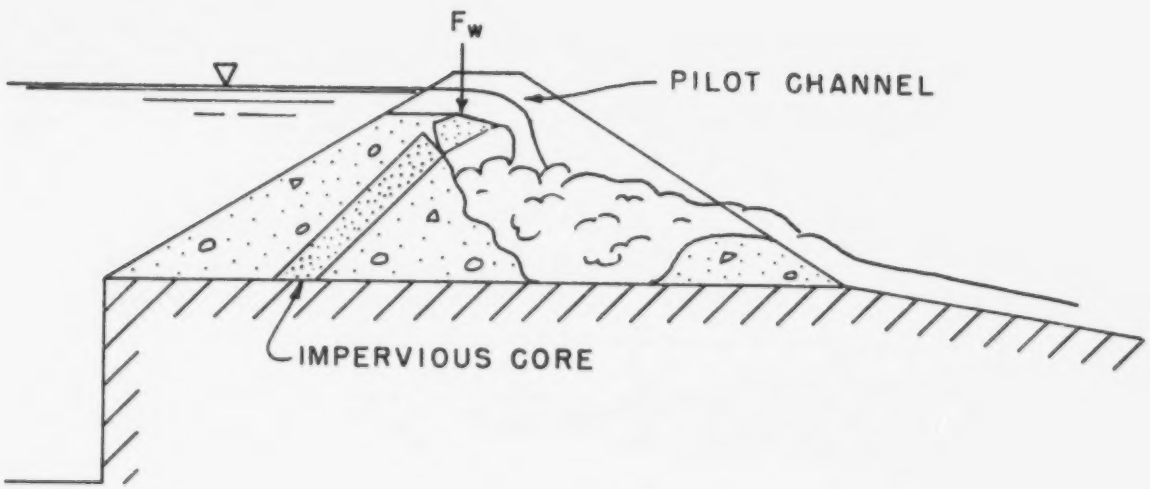


Figure 2-16. - Flow through the pilot channel showing the failure mode of the impervious core.

For structural similitude,

$$\left(\frac{\gamma L}{\gamma E}\right)_m = \left(\frac{\gamma L}{\gamma E}\right)_p \quad (9)$$

If $\gamma_m = \gamma_p$ then

$$\frac{E_p}{E_m} = \frac{L_p}{L_m} = L_r \quad (10)$$

The ratio of the moduli of elasticity must equal the model scale ratio. However, it is difficult to find a model core material that has a modulus of elasticity low enough to satisfy this ratio and also maintain a seal. Therefore, a mixture of 10 percent clay and 90 percent sand with a modulus of elasticity approximately equal to that of the prototype was used. The thickness of the clay core portion of the model was reduced instead of using a material with a lower modulus of elasticity.

Because the model modulus of elasticity (E_m) was too large, the core thickness needed to be reduced to compensate. A clay core reduced in thickness (1/3 of geometric scaling) was used in most of the tests. This thickness resulted from computing the correct moment of inertia in the model to compensate for the modulus of elasticity being too large in the model. The remainder of the core thickness was built with sand sprayed with a stabilizing agent.

Model Measurements

During each test several measurements were made to document the washout.

1. Discharge was recorded at 3-second intervals. The measurement weir flow was subtracted from the total (initial) flow to obtain the flow through the breach.

2. The reservoir level was recorded continuously.

3. Flags were placed at 1 foot intervals on the top of the embankment, and a grid pattern was painted on the downstream face of the embankment. The lateral erosion rates (E_l) were recorded by noting the time that the erosion reached each flag. These rates were checked by viewing the videotape.

4. Each test was filmed using videotape cameras, still photographs, and slides. One video camera was located downstream from the embankment, and the other filmed the washout through the acrylic plastic end wall.

5. The discharge, reservoir level, and time were recorded on magnetic disk with the aid of a microcomputer.

Model Results

The erosion process and lateral erosion rate are dependent on the geometric configuration of the embankment. Figure 2-17 shows the configuration of the model embankments and defines the symbols. Table 2-1 lists the values of each of the pertinent features for each test. These values are listed as a dimensionless ratio of the fuse plug height (H).

The rate of erosion was consistent for any one model configuration and flow condition (see figs. 2-18 through 2-25). The erosion rate varied as the model configuration or flow condition was changed. The first number in the regression equations, shown on each graph, is the lateral erosion rate in ft/min.

Before each test the reservoir level was held constant at a level below the pilot channel elevation, with a long adjustable weir. The test was begun by raising the water surface to a level equivalent to a 0.3 m (1 ft) water depth in the prototype pilot channel. The material downstream of the core eroded down to the base of the fuse plug. When the support was removed from beneath, a piece of the core broke off. This process then reoccurred until the material in the pilot channel was completely washed away. Figure 2-26 is a series of photographs illustrating the erosion process.

After the initial breach, the embankment eroded laterally. The flow eroded the face of the embankment downstream from the core in a steady, continuous manner. As the noncohesive material washed away removing support, pieces of the core broke off. Figure 2-27 is a schematic diagram illustrating how water flowed around the core and eroded the downstream embankment.

ANALYSIS

An analysis was made of the model results to determine the effect of the pertinent geometric and flow parameters.

Effect of Embankment Design Features

Core. - The role and effect of the impervious core was analyzed in the model study. A previous section on "Structural Similitude" discusses the structural modeling of the core. A clay core 1/3 of the thickness indicated by geometric scaling was used during most tests.

To assess the effect of the core thickness on the lateral erosion process, one test was conducted with a clay core thickness indicated by geometric similitude (test No. 2). This thickness was about 3 times more than that required for structural similitude. During this test the initial breach proceeded about the same as the other tests until the first core break, when the core overhung and did not break. The first break was assisted by manually breaking the clay, after which the washout process proceeded much the same as in the other tests. The lateral erosion rate was 0.463 m/min (1.52 ft/min). This erosion rate was only 2 percent less

Table 2-1. - Fuse plug model test data

Test No.	Date 1983	H (ft)	W/H	B/H	b/H	θ (deg)	T/H	t/H	L/H	p/H	h/H	Sand filter	D/J	D/H	E_1 (ft/min)
1	2-14	1.0	0.4	4.4	3.1	45	0.12	0.04	0.0	0.24	0.12	Yes	0.21	0.92	1.74
2	2-28	1.0	0.4	4.4	3.1	45	0.12	0.12	0.12	0.36	0.12	No	0.21	0.92	1.52
3	3-10	1.0	0.4	4.4	4.0	30	0.12	0.04	0.32	0.48	0.12	No	0.21	0.92	1.53
4	3-21	1.0	0.4	4.4	3.1	45	0.12	0.04	3.24	0.48	0.12	No	0.21	0.92	1.55
5	4-4	1.0	0.8	4.8	3.4	45	0.12	0.04	0.45	0.74	0.12	Yes	0.15	0.92	1.60
6	4-11	0.5	0.8	4.8	3.4	45	0.12	0.04	0.91	1.48	0.24	Yes	0.07	0.84	0.68
7	4-19	1.25	0.8	4.8	3.4	45	0.12	0.04	0.51	0.88	0.24	Yes	0.17	0.84	1.66
8	6-7	1.25	0.8	4.8	3.4	45	0.12	0.04	1.60	3.20	0.24	Yes	*0.15 0.12	*0.73 0.60	*1.43 0.63

1 ft = 0.3048 m
* The upstream water level (D) was lowered during test No. 7.

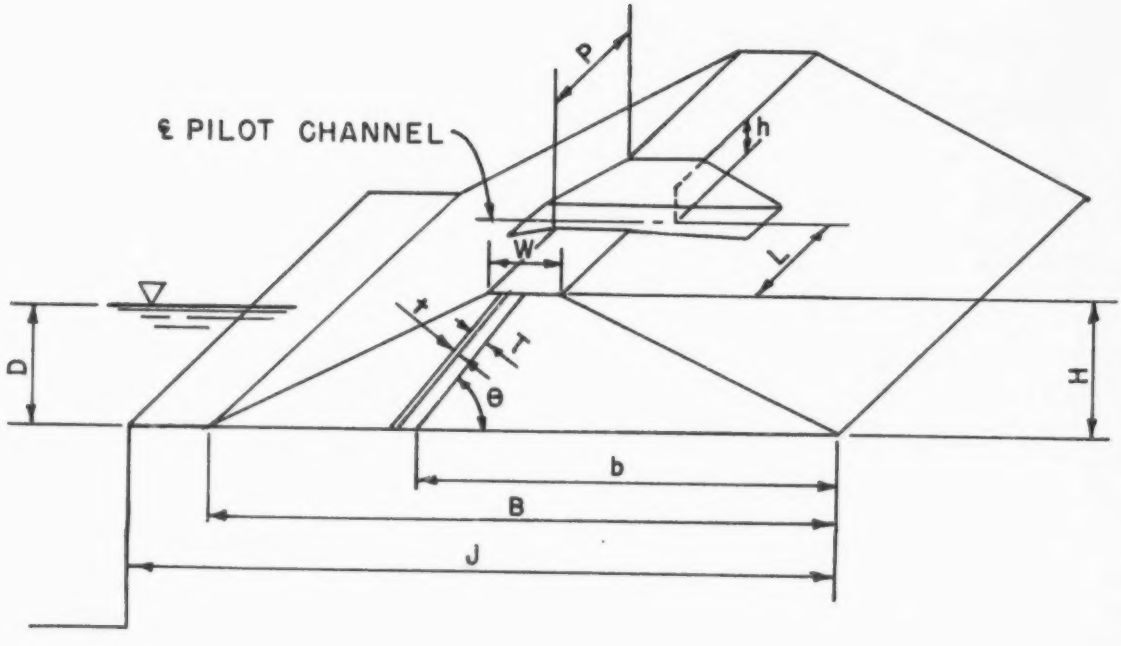


Figure 2-17. - Definition sketch of geometric features of model fuse plug embankment.

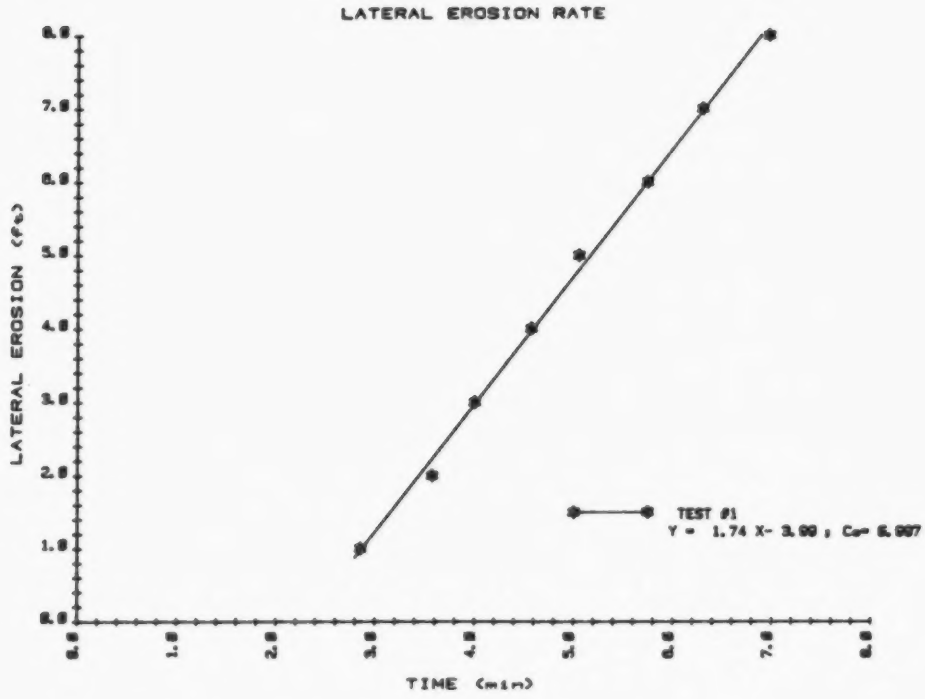


Figure 2-18. - Lateral erosion rate, test No. 1

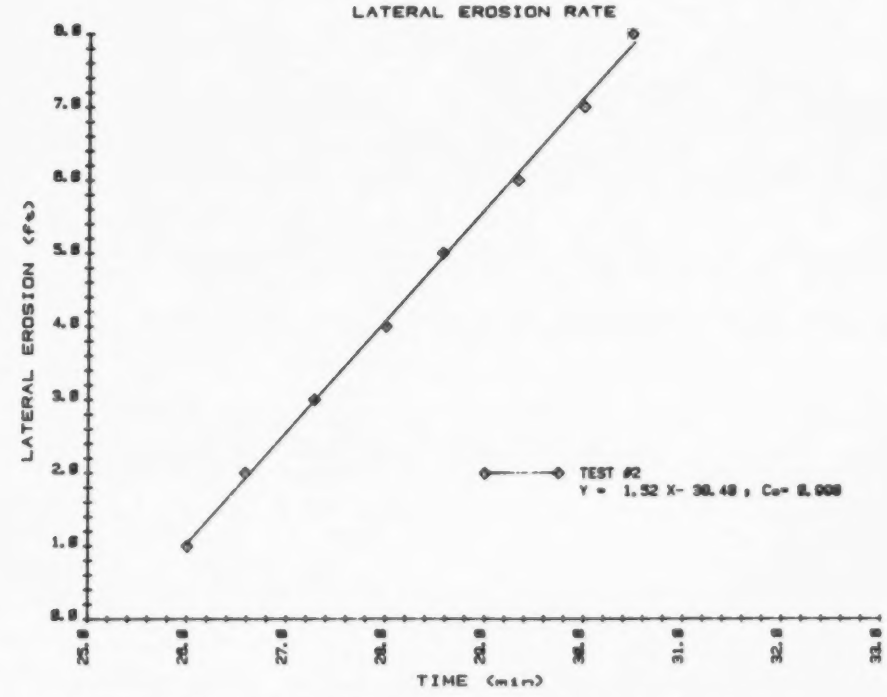


Figure 2-19. - Lateral erosion rate, test No. 2.

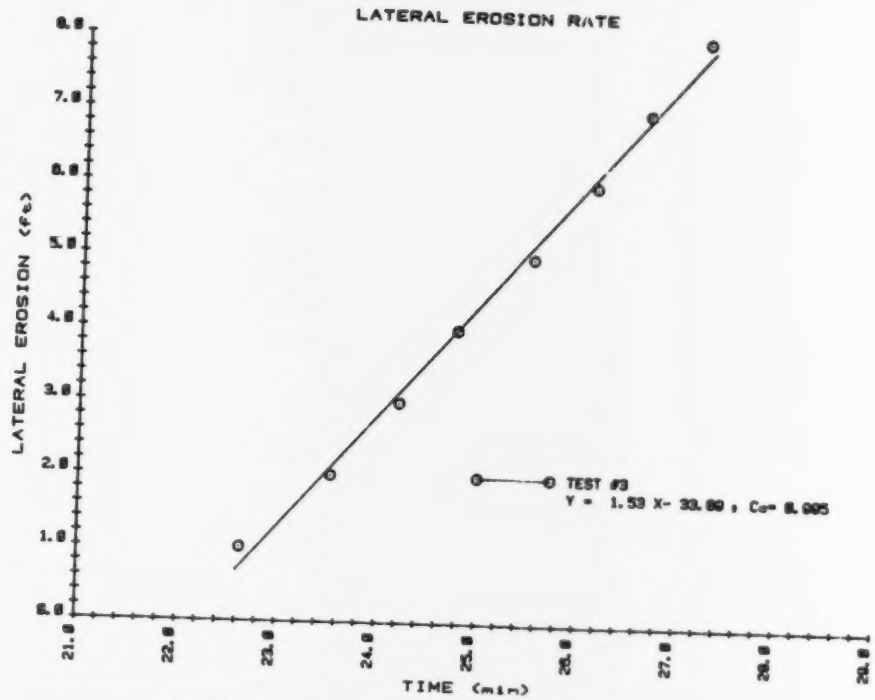


Figure 2-20. - Lateral erosion rate, test No. 3.

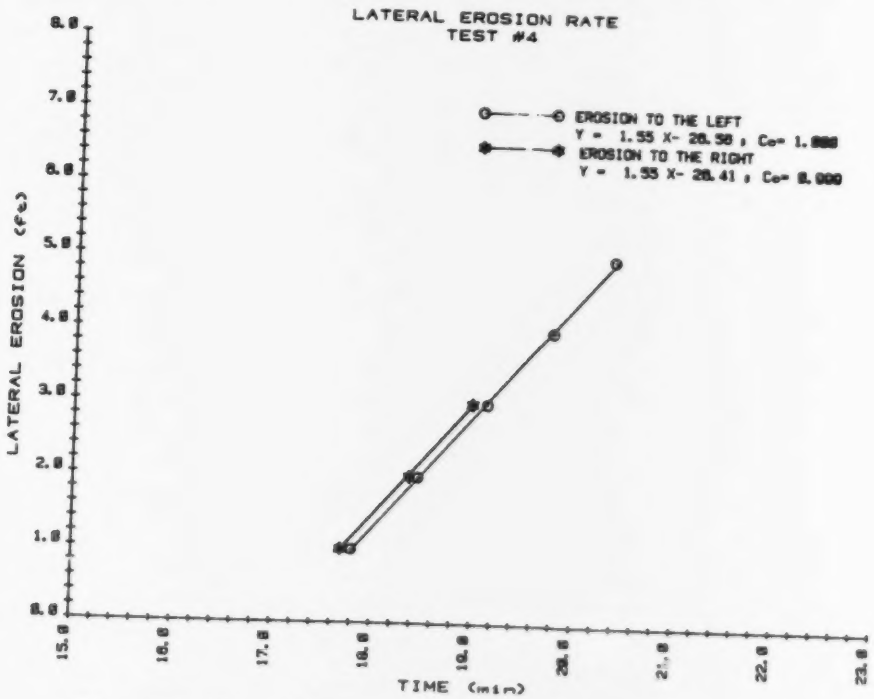

LATERAL EROSION RATE
TEST #4
LATERAL EROSION (Ft)
TIME (min)
EROSION TO THE LEFT
Y = 1.55 X- 28.58 ; Co= 1.000
EROSION TO THE RIGHT
Y = 1.55 X- 28.41 ; Co= 0.000

Figure 2-21. - Lateral erosion rate, test No. 4.

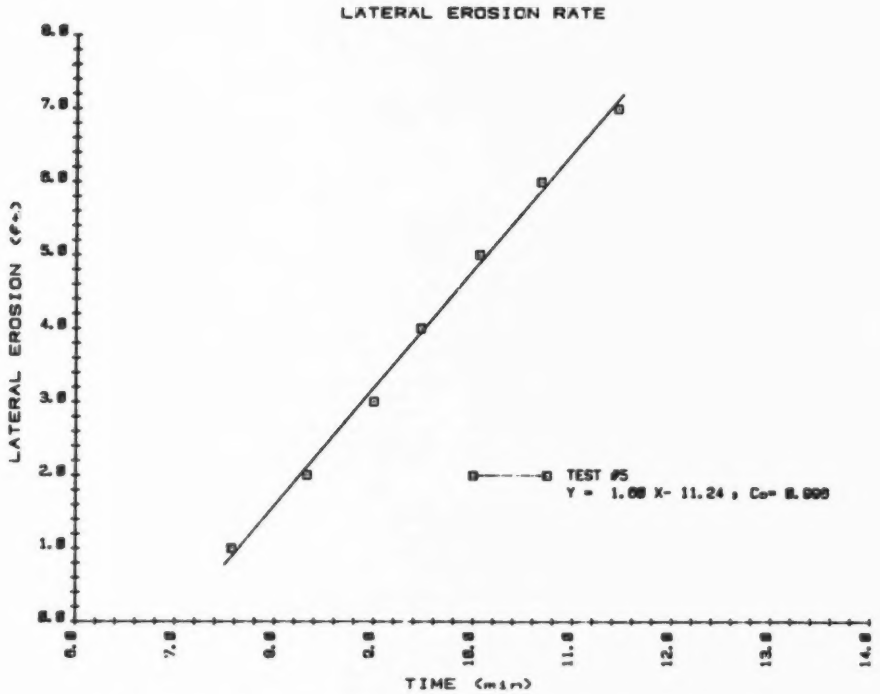


Figure 2-22. - Lateral erosion rate, test No. 5.

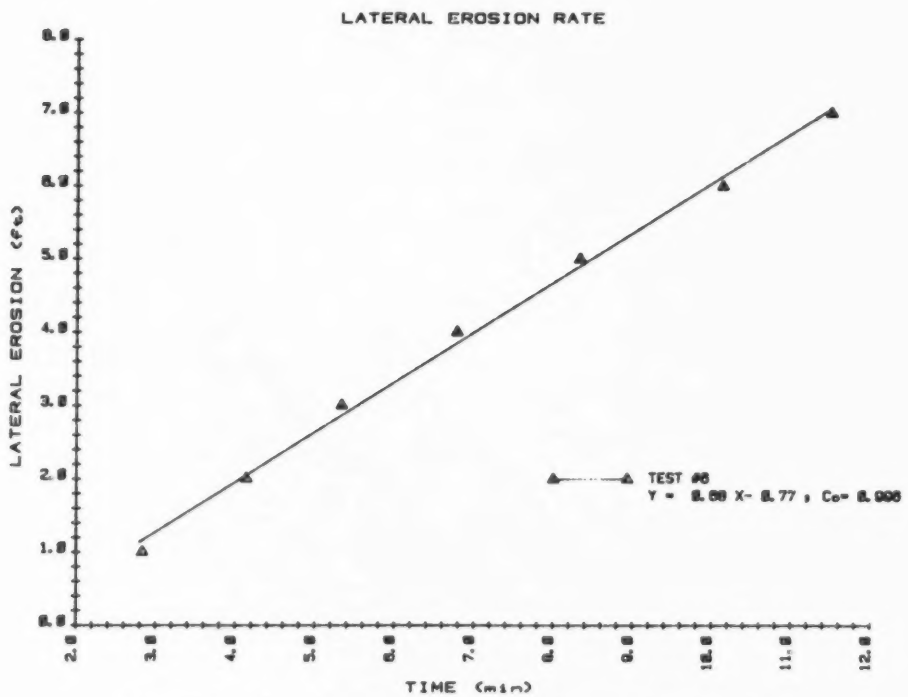


Figure 2-23. - Lateral erosion rate, test No. 6.

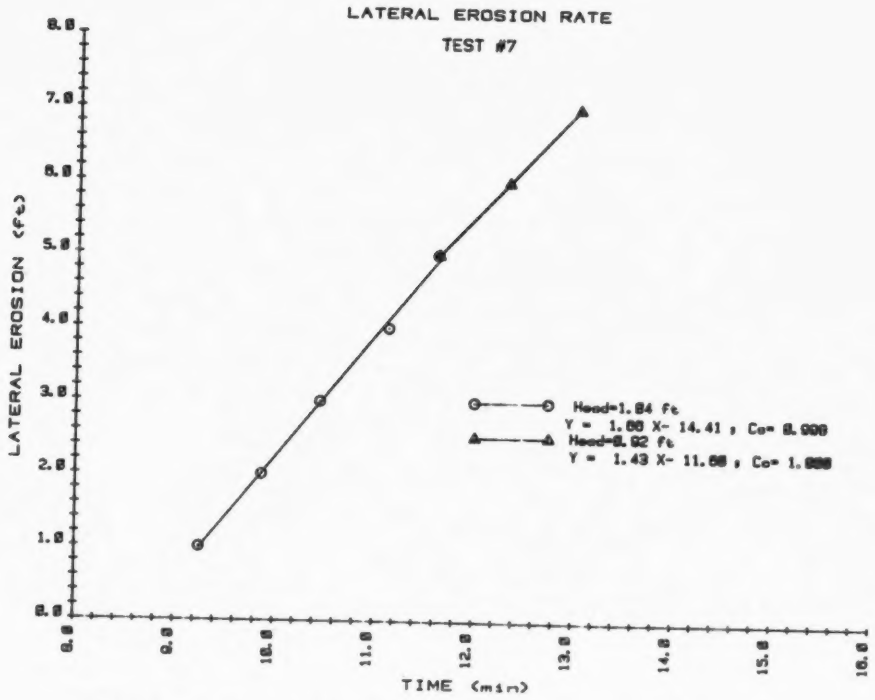


Figure 2-24. - Lateral erosion rate, test No. 7.

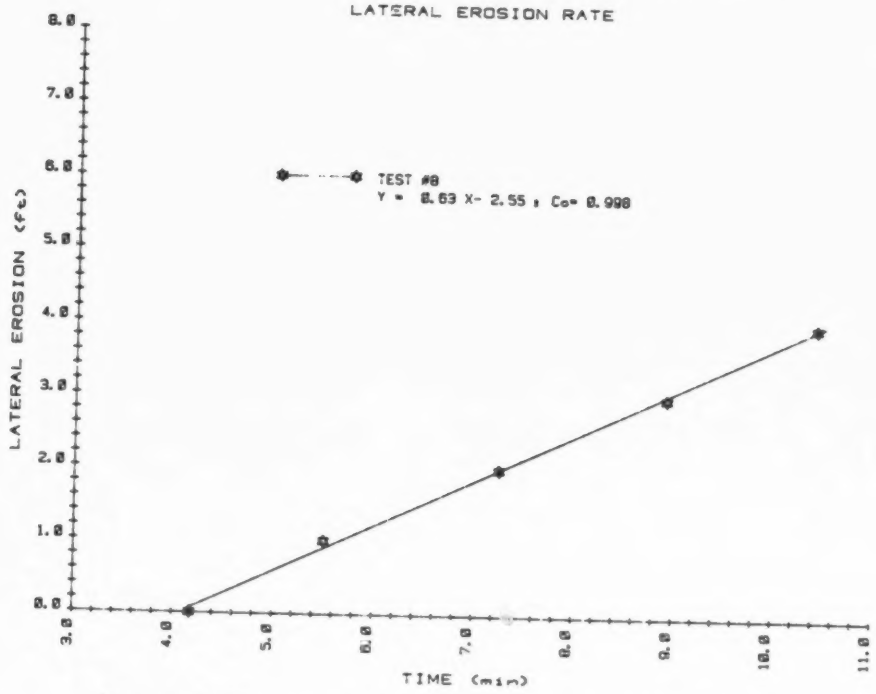
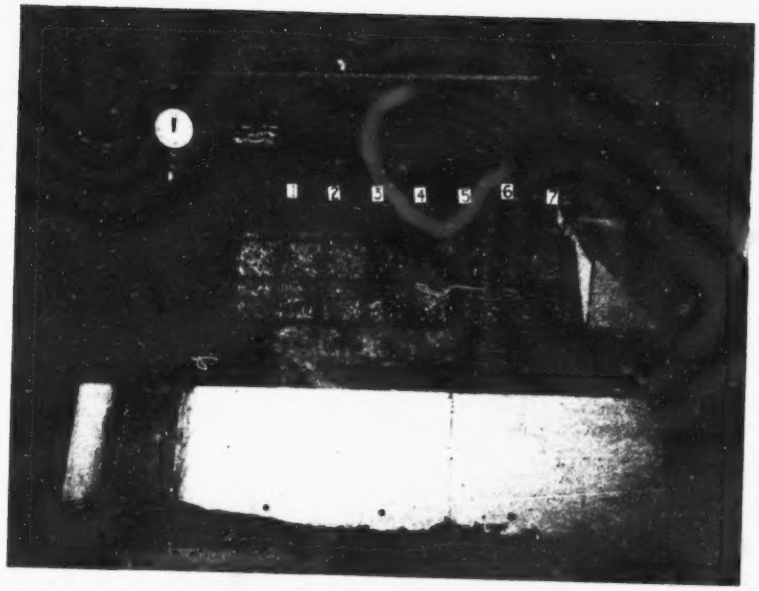
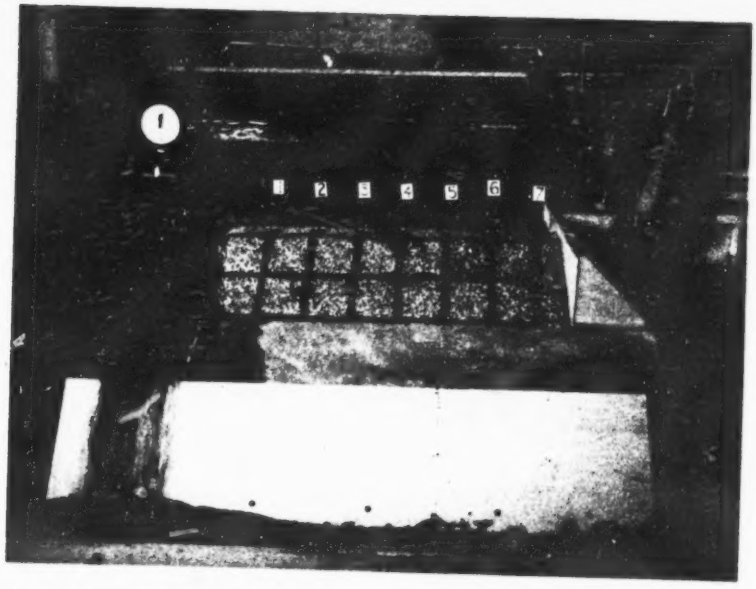


Figure 2-25. - Lateral erosion rate, test No. 8.

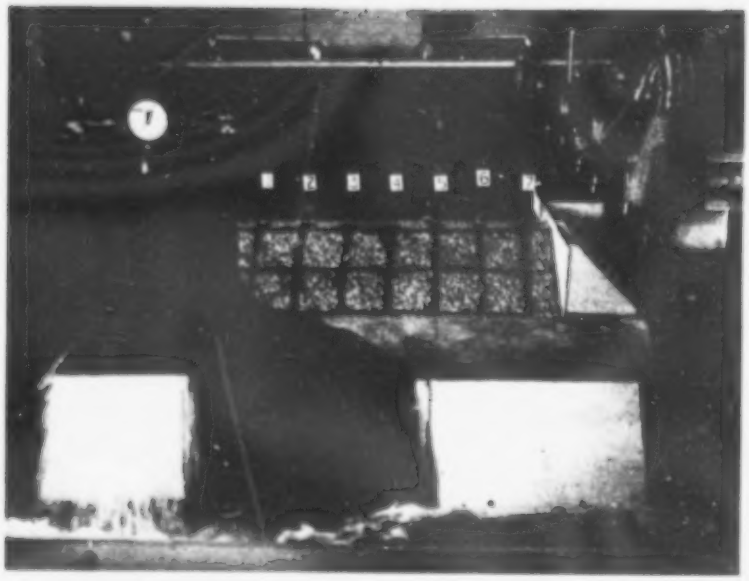


(a) Water flowing through the pilot channel over the clay core.

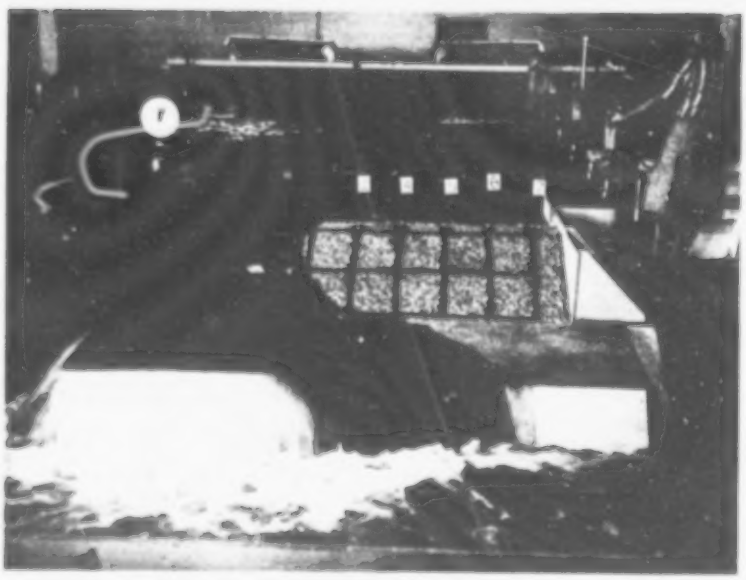


(b) After initial breach

Figure 2-26. - Photographs illustrating the washout process (sheet 1 of 3).

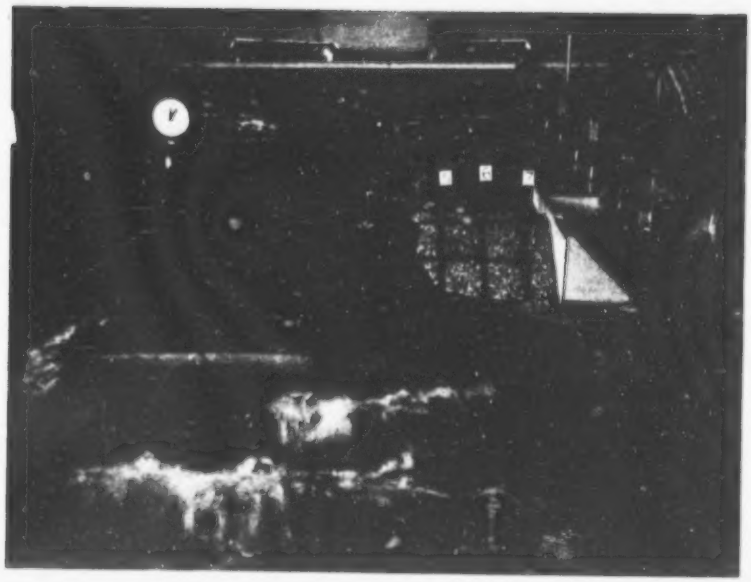


(c) The lateral erosion process is underway.

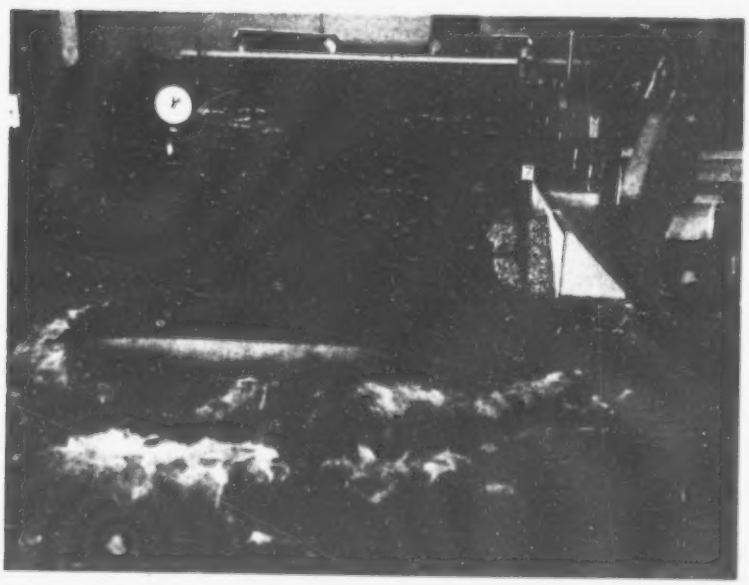


(d) The lateral erosion rate was determined by timing the erosion between flags.

Figure 2-26. - Photographs illustrating the washout process (sheet 2 of 3).



(e) The reservoir elevation was held constant by the long adjustable weir in the background.



(f) The erosion process is almost complete.

Figure 2-26. - Photographs illustrating the washout process (sheet 3 of 3).

than that in an identical test with a thinner clay core (test No. 4, E_1 = 0.472 m/min (1.55 ft/min)). During the lateral erosion process in test No. 2, the noncohesive material downstream from the core eroded at a constant rate and the core broke off in bigger pieces than in the other tests.

This test indicates that the lateral erosion rate is a function of the erosion rate of the noncohesive material and not a function of the core strength.

During test No. 3 the core was installed at an angle 30° above horizontal. The material downstream from the core was shielded by the core and the initial breach took longer. The lateral erosion rate was about the same. It was decided to use a core angle of 45° during future tests to prevent shielding of the downstream materials.

Pilot channel. - Various widths and positions of the pilot channel were tested. The location of the pilot channel did not have a noticeable effect on the lateral erosion rate. The erosion rate for test No. 4, with the pilot channel located near the center of the embankment, was the same in both directions, and about the same as a similar test with the pilot channel close to the end of the embankment.

The width of the pilot channel controls the amount of water passing through to initiate the breach. The model tests indicate that the pilot channel width (p) should be at least 1/2 of the fuse plug height ($p/H \geq 0.5$).

Sand filter. - Tests were run with and without the sand filter surrounding the main embankment downstream from the core. (See fig. 2-8 for embankment zoning.) It was found that the sand filter has a significant effect on both the initial breach and the lateral erosion. Without the filter the water flowing through the pilot channel infiltrates the downstream noncohesive material and partially saturates this zone, thus prolonging the breaching process. With the more permeable filter the water flowing through the pilot channel infiltrates down the face of the core. The pilot channel erosion process proceeds much more rapidly. The lateral erosion rate is significantly slower when the downstream sand filter is removed. Test No. 1 and test No. 4 have identical embankment designs, with the exception of the sand filter. The erosion rate without the sand filter is about 11 percent slower. The volume of the compacted sand and gravel zone is greater without the sand filter, since it also occupies the sand filter zone. The compacted sand and gravel requires more tractive force to move because these particles are larger and their sizes vary more than those in the sand filter material.

Size of embankment. - The relative size of the embankment was investigated in test No. 5. The width of the fuse plug crest was doubled, thereby increasing the area of the eroding face of the embankment. The erosion rate for Test No. 5 was about 8 percent less than for test No. 1. This is about the same percentage as the increase in the area of the downstream compacted sand and gravel zone.

Hydraulics of Flow Through the Opening

Broad-crested weir. - Flow through the opening is the same as flow over a broad-crested weir. Broad-crested weir flow is dependent on the depth of water above the crest (D) and the length of the crest (J). In the range $0.08 < D/J < 0.5$ [7], flow over a horizontal crest will be in the broad-crested weir flow range. Figure 2-28 shows broad-crested weir flow profiles for test Nos. 6, 7, and 8. For broad-crested weir flow the top surface and flow streamlines become parallel with the horizontal crest. The flow discharge is controlled by the critical flow depth near the end of the crest [8].

Discharge coefficients. - The following equation expresses discharge over a weir as a function of water depth.

$$Q = CLH^{3/2} \tag{11}$$

For critical depth, $d_c = (2/3)H$, and using the Froude number at critical depth:

$$F = 1 = \frac{V}{\sqrt{gd_c}} \tag{12}$$

Equation 12 can be expressed as

$$q = g^{1/2}(2/3)^{3/2}H^{3/2}$$

or

$$q = 3.09\ H^{3/2}$$

where the unit discharge, $q = Vd_c$

The coefficient (C) in equation 11 is 3.09.

For a broad-crested weir, the critical depth point is actually slightly upstream from the downstream end of the crest, and losses reduce the theoretical value of C slightly. Empirical data [7] indicate that in the broad-crested weir range $0.08 \leq D/J \leq 0.5$ the theoretical discharge coefficient is reduced by a factor of 0.848. Therefore, equation 11 would be:

$$Q = (0.848)(3.09)LH^{3/2}$$

or

$$Q = (2.62)LH^{3/2} \tag{13}$$

Discharge coefficients measured during test No. 7 are shown on figure 2-29. These coefficients were computed from the discharge and water depth measurements made during the test. The discharge coefficients increase to a maximum value of 2.78 for a breach length (L) of 1.2 to 1.8 m (4 to 6 ft). The coefficient drops slightly, to C = 2.62, when the embankment is completely washed away. This value matches the empirical value for a broad-crested weir

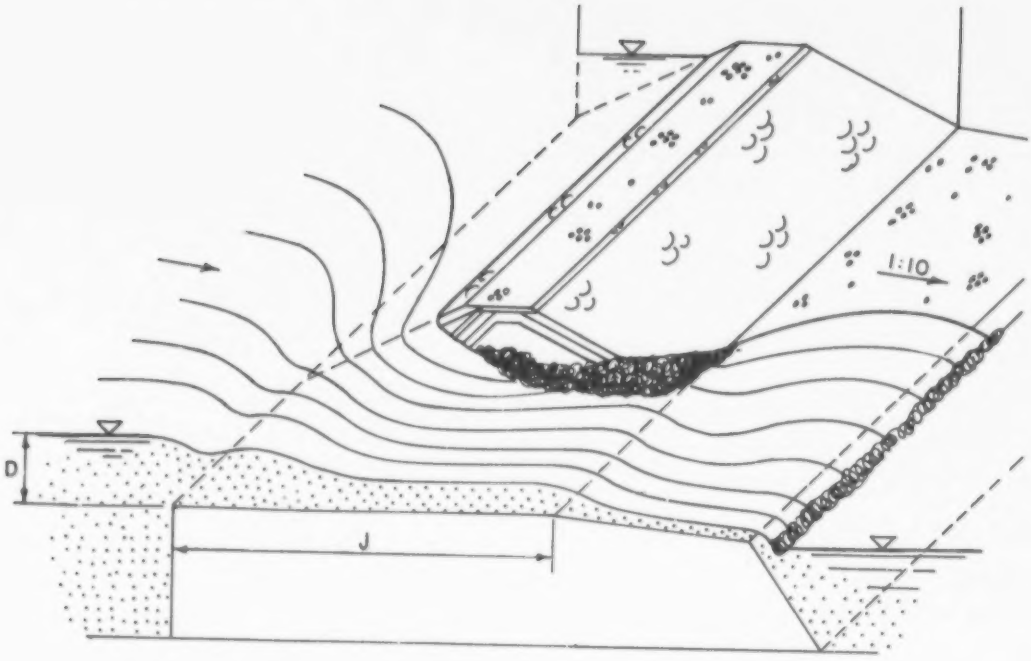


Figure 2-27. - Schematic of lateral erosion process. The water flows across the face of the embankment, around the core, and erodes the noncohesive material downstream from the core.

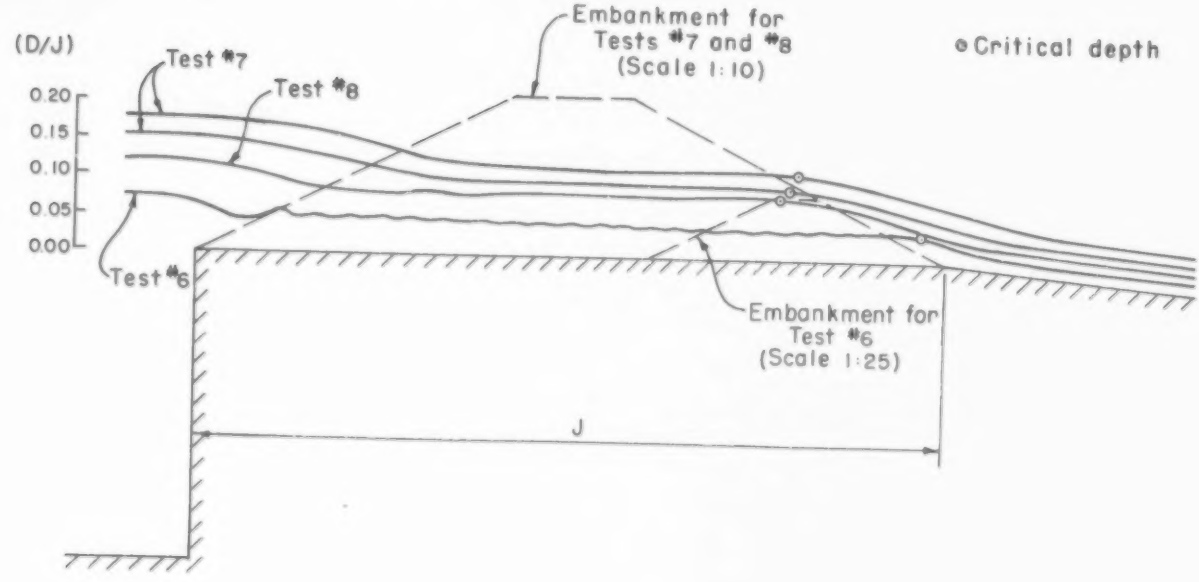


Figure 2-28. - Broad-crested weir flow profiles, test Nos. 6, 7, and 8.

(equation 13). The higher discharge coefficient during the washout can be attributed to a longer effective weir length caused by the flow coming around the face of the embankment (fig. 2-27). During test No. 6, with a relatively longer crest length, J (see fig. 2-28), the discharge coefficient during the washout was as high as 3.1.

These discharge coefficients can be used in a computer flood-routing program. The lateral erosion rate is a function of the depth to crest length ratio (D/J) and the depth to fuse plug height ratio (D/H). Tests Nos. 6 and 7 have the same D/H ratio (0.84), but different D/J ratios (see fig. 2-28). The result is a different erosion process. For (D/J) > 0.12, the flow surface was still drawing down as it reached the embankment. This caused a longitudinal vortex along the face of the downstream compacted sand and gravel that accelerated the erosion rate. For D/J < 0.12 the flow was parallel to the crest as it reached the embankment. The erosion was similar to streambank erosion with no vortex to aid the erosion process.

The lateral erosion rates are shown as a function of water depth and crest length on figure 2-30. This figure illustrates that the erosion rate is a function of the water depth squared for D/J < 0.12. For D/J > 0.12, the erosion rate is accelerated.

This analysis illustrates that the length of crest (J) has a major effect on the erosion process. It also illustrates that when it is desirable to slow the fuse plug erosion process, the length of the approach channel could be increased.

Projection to Prototype

The results from this study can be used to predict the behavior of a prototype fuse plug embankment. The discharge coefficients discussed in the previous sections can be used to predict flow through a given opening size.

The lateral erosion rate for a given embankment design and flow depth can be predicted from the model tests. Figure 2-31 shows the erosion rates for an embankment with dimensions given in table 2-1 for test No. 1. The flow depth was determined by the pilot channel design, with 0.6 m (2 ft) of freeboard and 0.3 m (1 ft) of water depth, regardless of the embankment height. Equation 14 can be used to estimate erosion rates for embankments of this configuration that are from 3 to 9 m (10 to 30 ft) in height.

$$E_l = 14.6\ H + 158 \qquad (14)$$

where E_l is in ft/hour, and H is in feet.

If the configuration of the embankment or the flow condition is changed, the erosion rate given by equation 14 would be changed accordingly. For example, if the area of the downstream compacted sand and gravel section is increased by 10 percent, the erosion rate predicted by equation 14 should be reduced by 10 percent. If the water depth or crest length (J) are different from those on figure 2-31, the erosion rate would be adjusted using figure 2-30.

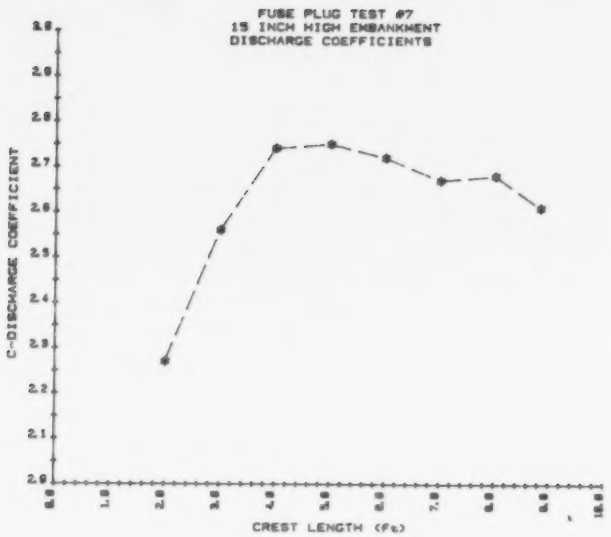


Figure 2-29. - Weir formula discharge coefficients, test No. 7.

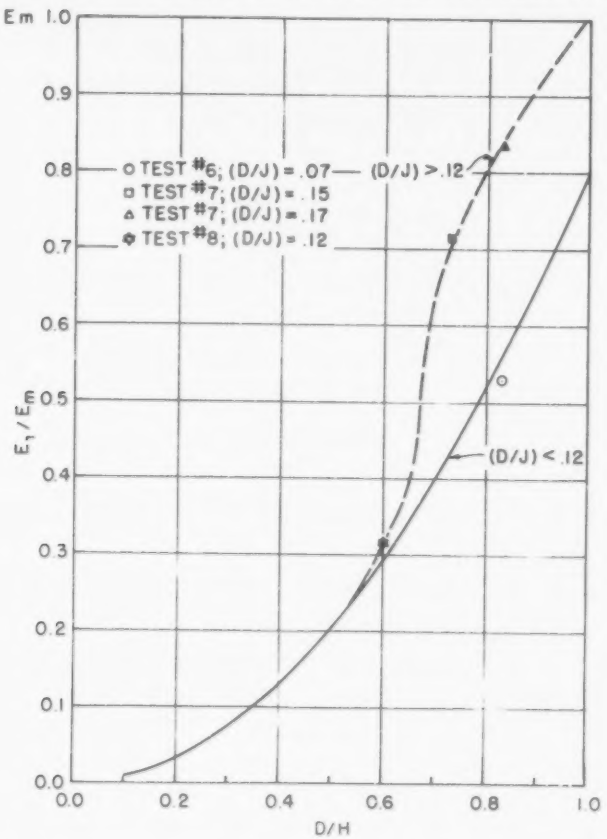


Figure 2-30. - Lateral erosion rate as a function of water depth and crest length.

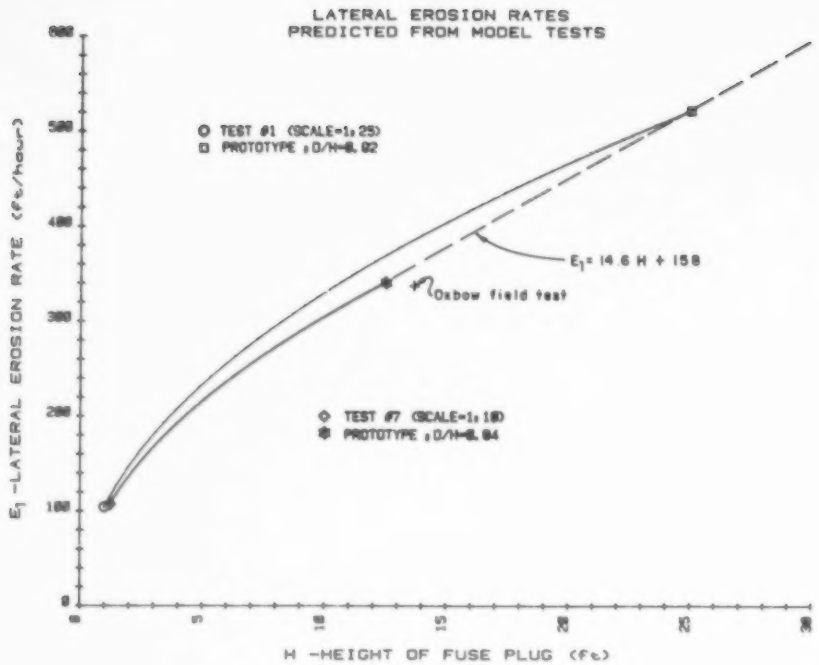

LATERAL EROSION RATES
PREDICTED FROM MODEL TESTS
O TEST #1 (SCALE=1:25)
□ PROTOTYPE ; D/H=0.82
◇ TEST #7 (SCALE=1:18)
◆ PROTOTYPE ; D/H=0.84
$E_1 = 14.6\,H + 158$
Oxbow field test
E_1 -LATERAL EROSION RATE (ft/hour)
H -HEIGHT OF FUSE PLUG (ft)

Figure 2-31. - Lateral erosion rates predicted from model tests.

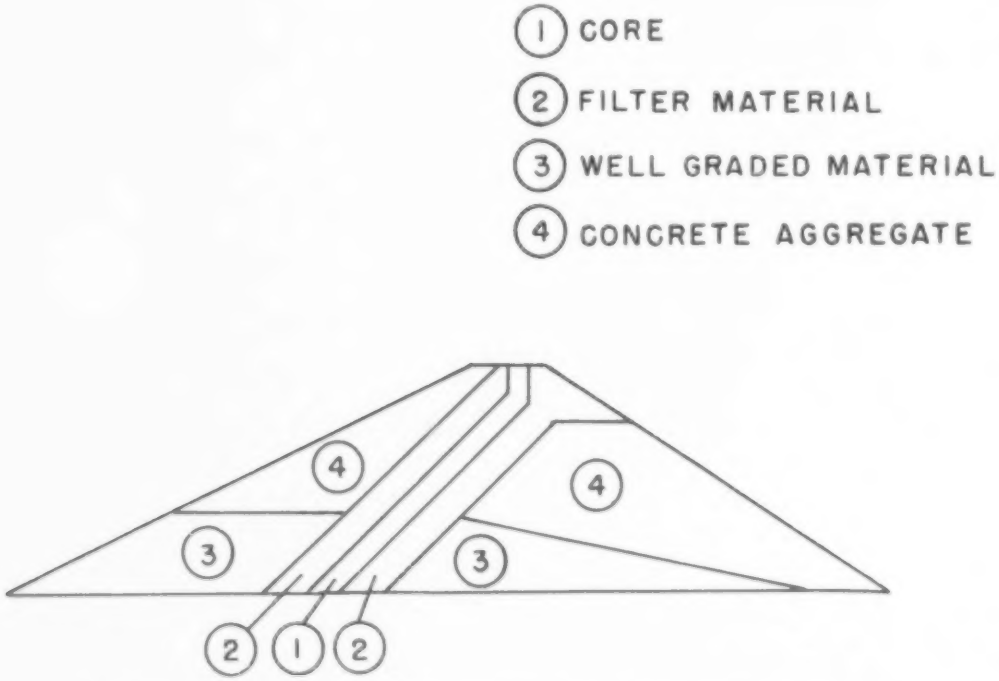


Figure 2-32. - Embankment design for Oxbow field test.

Comparison to Oxbow Field Test

The only data available on a prototype-sized fuse plug in operation are for a field test performed as part of the design of a fuse plug control for an auxiliary spillway on the Oxbow Project on the Snake River in Idaho. The 1/2 scale field test of an 8.2 m (27 ft) prototype embankment showed the same pilot channel breach and lateral erosion process indicated by this model study. The following are the geometric and flow parameters for the Oxbow field test (as defined on fig. 2-17).

Table 2-2. - Oxbow field test data

H (ft)	W/H	B/H	b/H	θ	T/H	L/H	W/H	h/H	S.F.	D/H	E_1 (ft/min)
13.5	0.37	4.12	2.59	45°	0.11	0.37	0.81	0.15	Yes	0.93	5.6

The Oxbow embankment design [1] is shown on figure 2-32. The gradation curve for the Oxbow zone 4 (concrete aggregate) was similar to that of zone 3 (compacted sand and gravel) in this study (fig. 2-11).

Although the embankment design for the Oxbow field test was slightly different from the embankments tested in this model study, the erosion rate was close to the erosion rate predicted by equation 14 (fig. 2-31).

During the later part of the Oxbow field test the entire embankment downstream from the sand filter was zone 3 (well graded) material. The erosion rate reduced from 102 to 22 m/hr (336 to 72 ft/hr) demonstrating the importance of material gradation.

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FLEXIBLE MEMBRANE

EMERGENCY SPILLWAY

by

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January 1984

Applied Sciences Branch - Division of Research and Laboratory Services
Concrete Dams Branch - Division of Dam and Waterway Design
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Engineering and Research Center

DENVER, COLORADO

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ABSTRACT

Present widespread concerns over the inadequacy of the emergency spillway capacity of existing dams and the need for low cost alternatives to meet new inflow requirements have lead to the search for new approaches. One attractive approach is the use of a buried, flexible membrane-lined emergency/auxiliary spillway. The USBR (Bureau of Reclamation) has initiated an investigation, which will cover the design, construction, and use of membrane materials appropriate for this engineering approach. As part of this study, a 36-mil hypalon flexible membrane-lined spillway will be constructed at the 20-foot-high Cottonwood Dam No. 5, a part of the Collbran Project located near Grand Junction, Colorado.

SUMMARY

With growing concern over inadequate emergency spillway capacity for embankment dams, low-cost alternatives are being sought. The use of flexible membrane-lined emergency spillways appears to have considerable merit. The membrane industry in the United States and abroad now offers a wide range of materials for consideration. Although limited work has been done by others in this field, the results appear promising.

The USBR (Bureau of Reclamation) has begun an investigation on the use of flexible membranes as a low-cost means of constructing emergency or auxiliary spillways for a variety of low-head structures, including embankment dams. With increased experience and knowledge, consideration can be given to higher head structures. The purpose of this investigation is to define design and construction guidelines and materials requirements.

A field study has been started and construction is expected to be completed in 1985. It will involve installation of a 262-foot-long, 36-mil hypalon flexible membrane lining on a spillway at Cottonwood Dam No. 5. This dam is located near Grand Junction, Colorado, at an elevation of 10,000 feet. A study program will follow the installation to evaluate the design, field performance, and materials used.

PROBLEM

On September 30, 1981, the Corps of Engineers discovered, in their inspection of non-Federal dams program, that nearly 3,000 dams were unsafe, requiring some remedial action. Of these, 81 percent were deficient because their spillways were too small to pass the estimated maximum floods. This reflects the difference between present-day design flood criteria, contained in the "Recommended Guidelines for the Safety Inspection of Dams" [1]*, and the criteria in vogue at the time the dams were constructed.

Embankment dams are particularly sensitive to failure caused by overtopping, both during construction and while in service. However, inadequate spillway capacity is not the only cause of overtopping failure. There have also been many cases where dams were overtopped because of gate failure.

These potential hazards can be avoided by adding an emergency spillway with the required discharge capacity. However, in many situations the

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* Numbers in brackets refer to entries in the Bibliography.

cost for a conventional concrete-lined spillway, or even a rockfaced compacted-earth spillway, would be prohibitive.

PROPOSED SOLUTION

An attractive approach is to provide a membrane-lined emergency spillway. The flexible membrane would be covered with soil until it is needed. At the beginning of emergency spillway operation, the soil would be washed away, and the membrane lining would carry the flow, protecting the embankment from erosion.

Significant advances have been made in recent years in the manufacture of flexible membrane materials that are suitable for a wide range of water resources engineering work. Therefore, many excellent materials are now available. Much work has been done to identify the important properties of these materials. Laboratory testing, field studies, and observations of these materials in place have provided guidance for the selection of durable materials.

A search of the literature did not turn up reports on the use of flexible membranes for emergency or normal spillways at embankment dams. However, one report suggested that some encouraging work was being done in France, and another study, from the U.S.S.R., concluded that flexible, or soft, spillways should be studied further.

INVESTIGATION

Because the use of a flexible membrane in the low-cost auxiliary spillway appeared to have merit, a study of modest scope was begun by the USBR. The investigation started with an evaluation of the feasibility of various applications for low-head structures. Locations where the consequences of failure would not be serious were given primary consideration. Some potential applications include:

- The many low-head earth dams of the BIA (Bureau of Indian Affairs), NPS (National Park Service), and other Department of the Interior agencies for which the USBR has some responsibility
- The low-head structures among the 50 to 60 percent of USBR dams, inspected to date, that have inadequate spillways
- New low-head earth dams
- Low-head dikes on large reservoirs
- Saddles suitable for emergency overflow where erosion could be a problem

- Side channels for low-head embankments
- Canal wasteways
- Diversion structures
- Drop structures
- Improvements to existing emergency/auxiliary spillways

The primary objective of the research is to develop design procedures, materials specifications, construction procedures, and cost data to assist in the selection, design, and construction of flexible membrane or geotextile emergency/auxiliary spillways for low-head structures.

The study is to include investigation of the following:

- The durability of materials with respect to aging and during passage of flood flows
- Structural design
- Hydraulic design
- The effects of permeability/impermeability with respect to uplift
- Cost data including cost comparisons
- Identification of limits of usefulness: heights, maximum water velocity, effects of drag on membranes, and the effects of membrane wrinkles on flow
- The consequences of failure
- Soil and vegetation (root) effects
- The ability to pass debris during floodflow
- Means of handling vehicular traffic
- Materials requirements

The study is currently focusing on the general design and related material requirements for low-head application. From this, some specific design and construction concepts will be developed. At the same time, limited field studies will help identify the advantages and specific areas needing more study. One field study has been started at Cottonwood Dam No. 5, located in western Colorado.

FIELD STUDY

The construction of a flexible membrane emergency spillway at Cottonwood Dam No. 5 is an important part of the present study. This small earth dam, which is being rehabilitated, offered an excellent opportunity for the field study. Some of the questions that should be answered from this study are:

1. When membranes are used on slopes, what effect does drag caused by the water velocity component have on the membrane? How does drag affect the membrane's tensile properties at its support, and does drag cause the uplift of the sheet from its foundation?

2. What are the effects of abrasive sands and materials on the membrane?

3. What is the most efficient method of anchoring the membrane along the sides and in the transverse direction?

4. What would be the minimum depth of cover material required to protect the membrane and the best type of cover material to use?

5. What effect does high water velocity, 30 to 40 ft^3/s, have on the membrane when the sheet is wrinkled after placement?

6. Are special foundation treatments needed before the membrane is placed?

7. What are the effects of aging on the durability and permeability of membranes?

8. How is the membrane affected by different soil types and vegetation (root) growth?

Cottonwood Dam No. 5 is one of 17 small private reservoirs that were constructed on Grand Mesa, near Grand Junction, Colorado. These reservoirs, which are filled during the spring runoff, were to regulate the runoff from small streams. This stored water is released on demand for irrigation. These reservoirs have been incorporated in the Collbran Project, which was developed to furnish both water and hydroelectric power. A Bureau of Reclamation SEED (Safety Examination of Existing Dams) report recommended that Cottonwood Dam No. 5 be breached and reconstructed. This recommendation provided the opportunity for the implementation of the flexible membrane emergency spillway study. The earth dam is to be 450 feet wide and 19 feet high.

DESIGN CONSIDERATIONS

The spillway design is shown on figure 3-1. A summary of the spillway design considerations is listed below.

Structural Design

1. The dam is to be rebuilt to elevation 10,004.0 before construction of the emergency spillway.

2. The spillway is alined to pass through the more plastic materials on the right abutment to provide additional erosion protection if needed.

3. Grade sills. Two grade sills are to be provided: one at the upstream end of the membrane liner to provide flow control and prevent piping; the other at the downstream end to prevent head-cutting, back into the spillway.

4. Anchorage of flexible membrane

   a. At grade sills - attached to concrete with redwood furring strips and nails to prevent separation from the grade sill and to distribute the load evenly across the sheets.

   b. Along the sides and transverse joints - edges of liner placed in trenches, which are subsequently backfilled and compacted.

5. Joints

   a. Splicing at this site is impractical.

   b. No longitudinal joints are allowed. This prevents stress buildup in one sheet from being transferred to another.

   c. Each sheet is to overlap its adjacent downstream sheet by approximately 5 feet to provide adequate protection of the compacted backfill and anchor the downstream sheet.

   d. Two-foot minimum overlap is to be provided on the horizontal curve.

6. Protective cover. Six inches of noncohesive material is to be provided over the flexible membrane to protect it from foot, animal, and vehicle traffic. Materials will be dredged from Cottonwood Lake No. 1, which is expected to wash away during spillway operation.

7. Alinement. The alinement shall be chosen so that there are no discharges along the toe of the dam.

Hydraulic Design

1. Inflow design flood = 100-year flood.

2. Design discharge = 40 ft^3/s.

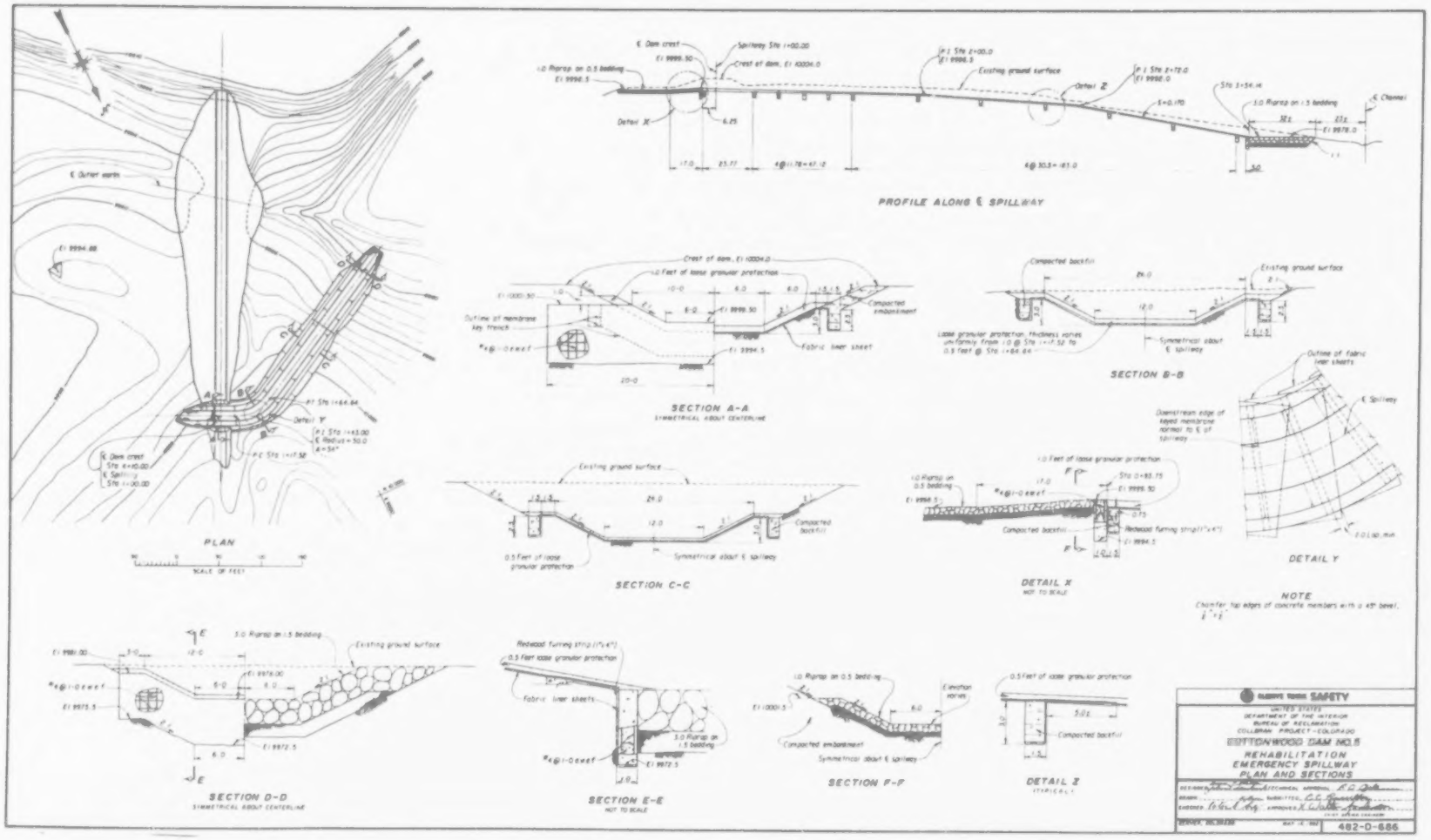

Figure 3-1. - Cottonwood Dam No. 5 - emergency spillway, plans and sections.

3. Flow conditions. Flow passes through the critical depth at the upstream grade sill; therefore, the flow is supercritical over all areas protected by the flexible membrane liner.

4. Channel

a. Bottom width = 12 feet, with 2:1 side slopes.

b. Depth = 3 feet (to provide freeboard). This is based on wave action in the horizontal curve.

5. The assumed Manning's number, "n" = 0.025 for the protective cover in place, and "n" = 0.015 for the flexible membrane.

6. Maximum channel velocity = 14.5 ft/s.

7. Energy dissipation is to be provided by a natural hydraulic jump, which should form over the downstream riprap protection.

8. Riprap is sized to resist movement caused by velocities associated with the design discharge

Physical Properties of Membranes

The desired physical properties of flexible membranes for applications in the emergency spillways include the following:

- High tensile strength and flexibility
- High puncture and abrasion resistance
- Good impact tear resistance
- Good weatherability
- Immunity to bacterial and fungus attack
- Ease of welding seams in the field

At the present time there are two types of lining materials that may be suitable for this application. These are the fabric-reinforced materials, such as hypalon and CPE (chlorinated polyethylene); and HPDE (high density polyethylene) materials. A few of the more important physical properties of these lining materials and their approximate costs are listed below.

	Reinforced materials	HDPE materials (includes HDPE/ alloy materials)
Available thickness, (mils)	30, 36, 45, 60	20, 30, 40, 60, 80, 100
Tensile strength, (lbf/inch width)	200 to 300	60 to 150

	Reinforced materials	HDPE materials (includes HDPE/ alloy materials)
Tear strength, (lb)	30 to 100	100 to 500
Approximate cost, ($/ft^2)	0.45 to 0.75	0.40 to 0.90

Since different test methods and sample sizes are used to determine tensile and tear strengths, and puncture resistances, the values above are not directly comparable between the two types of lining materials.

For a test at Cottonwood No. 5, field seaming would be impractical because of the remote location of the dam. Therefore, only reinforced sheets will be used at Cottonwood Dam No. 5, since they can be prefabricated for transport to the jobsite.

The material selected for the field study is 36 mil reinforced hypalon sheet, fabricated to 38 by 40 feet and 38 by 23 feet. This material met the specifications shown in table 3-1.

The construction considerations cover the foundation preparation, cutoff wall construction, riprap and bedding, and the flexible membrane spillway lining.

Before the flexible membrane lining is installed, the subgrade will be inspected to ensure that it is free of depressions, wet areas, and any rocks or other sharp materials, objects, or debris that can puncture the lining.

During all periods of shipment and storage, the membrane lining is protected from direct sunlight, temperatures greater than 140 °F, mud, dirt, dust, and debris. To the extent possible, the membrane is maintained wrapped in a heavy-duty protective covering, or covered at the jobsite until it is installed. Before installation, the membrane lining will be inspected for defects, such as rips, tears, and holes; and it will be repaired accordingly.

The membrane lining will be installed and anchored as shown on drawing 482-D-686 (fig. 3-1). Care will be taken to ensure that the lining is placed in the spillway in a slackened condition, so that it will conform to the contours of the subgrade without being taut when covered with the protective earth cover material. The membrane liner sheets will be installed with the long dimension parallel to the centerline of the spillway channel and overlapped 5 feet in the direction of flow, as shown on the drawing. Backfill should be placed and compacted carefully in the anchor trenches so that the lining is not damaged.

Table 3-1. - Material properties

Property	Test method	Value
Gage (nominal)		36
Piles reinforcing		1
Thickness (mils, minimum)	ASTM D 751	
1. Overall		34
2. Over scrim	Optical method	11
Breaking strength-fabric (pounds, minimum)	ASTM D 751 method A	200
Tear strength (pounds, minimum)	ASTM D 751 (modified)	
1. Initial		60
2. After aging		25
Low temperature (°F)	ASTM D 2136 1/8 in mandrel, 4 h, pass	-40
Dimensional stability (each direction percent change maximum)	ASTM D 1204 212 °F, 1 h	2
Volatile loss (percent loss maximum)	ASTM D 1203 method A, 30-mil sheet	0.5
Resistance to soil burial (percent change maximum in original values)	ASTM D 3083 30-mil sheet (modified)	
a. Unsupported sheet		
1. Breaking strength		5
2. Elongation at break		20
3. Modulus at 100% elongation		20

Table 3-1. - Material properties - Continued

Property	Test method	Value
Membrane fabric breaking strength	ASTM D 751 method A, procedure 1	25
Hydrostatic resistance (lb/in^2, minimum)	ASTM D 751 method A, procedure 1	250
Ply adhesion (each direction lb/inch width, minimum)	ASTM D 413 machine method type A	7
Factory Seam Requirements		
Bonded seam strength (factory seam, breaking factor, lb, width)	ASTM D 751 (modified)	160
Peel adhesion (lb/inch, minimum)	ASTM D 413 (modified)	Ply separation in plane of scrim or 15 lb/in
Resistance to soil burial (percent change maximum in original value)	ASTM D 3083 (modified)	
Peel adhesion		20
Bonded seam strength		-25

An earth cover will be placed over the flexible membrane lining in the spillway to the depth shown on the drawing (fig. 3-1). This earth cover material will be obtained from the stockpiled dredge material from Cottonwood Dam No. 1. Rocks, boulders, vegetable matter, brush, large roots, or other objectionable foreign matter will not be permitted in the earth cover material. This material will be spread by equipment pushing the adjacent in-place earth cover material. No equipment will be permitted directly on the membrane lining. The method and manner of placing the earth cover material on the lining will be subject to approval.

PROPOSED STUDY PROGRAM AT COTTONWOOD DAM NO. 5

To obtain the maximum benefit from the field study, the following study program will be followed:

Short-Term Items

1. During installation

   a. Obtain information for the selection of materials used.

   b. Note any possible improvements in the design concepts.

   c. Determine construction guidelines for future construction.

   d. Establish two or three survey control points for elevation and alinement.

2. Study the flow with the cover material in place

   a. Observe the movement of the cover material with flow.

   b. Determine the need for a fuse plug.

   c. Observe the density of the material before and after flow.

   d. Establish the depth of cover needed for flow requirements.

3. Study the flow without the cover protection

   a. Observe membrane behavior, movement, uplift, and stress.

4. Water requirements for testing

   a. Use of flashboards in the gate chamber.

   b. If additional water is required:

      (1) Release water from the upstream reservoir.
      (2) Sandbag test the spillway to permit surge flow.

Long-Term Items

1. Long-term weathering and compaction of cover material - how does this effect the flow?

2. Removal of membrane samples to study aging.

3. Damage to membrane lining from:

   a. Root growth.

   b. Animal hoof damage.

4. Study of the removal of the cover material after a period of weathering and the effect on the mebrane after a curing period.

5. An annual field inspection of damage to the membrane.

6. If a fuse plug is used, what are the weathering effects?

7. The effects of water velocity on the membrane.

8. Anchoring methods for membrane protection.

9. Crest sill construction and end sill construction.

   a. The best method to anchor and protect the membrane.

10. The best methods of patching the membrane, if necessary.

As of October 1983, the construction of the dam has progressed to the point where little additional work is required before construction of the spillway could begin. Because the dam is located at an elevation of 10,000 feet and in a remote location, the construction site is sensitive to the weather. It had been hoped that the spillway would be installed and first tested in 1982, however, heavy winter snows and summer rains have resulted in a 2-year postponement of the spillway construction.

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DAM CREST RAISING WITH REINFORCED EARTH RETAINING WALLS

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January 1984

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ABSTRACT

Safety evaluations of Bureau of Reclamation dams have resulted in the modification of several existing structures for flood events larger than that considered in the original design. In many cases, an economical method of modification is to raise the existing structure, providing additional storage capacity, rather than increase the spillway or outlet discharge capacities. At Lake Sherburne Dam the most economical solution to accommodating the updated PMF (probable maximum flood) was to raise the existing dam approximately 15 feet. Cost savings were realized by using the patented Reinforced Earth* retaining wall system to raise the dam. Conventional methods of raising the dam would have required considerable quantities of earthfill materials, modifications to existing concrete structures, and additional construction time. Major considerations in the designs included external and internal stability for static and earthquake loading conditions, service life, and water retention capability.

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* Reinforced Earth is a registered trademark of the Reinforced Earth Company, Arlington, Virginia.

INTRODUCTION

Lake Sherburne Dam, located in northern Montana, was constructed by the Reclamation Service, between 1914 and 1921, to provide reservoir storage for irrigation of project lands. The embankment is a rolled earthfill structure consisting of high density gravelly clay. Prior to modification (fig. 4-1), the dam had a structural height of 85 feet and crest length of 900 feet. The reservoir formed behind the dam had a total capacity of about 90,000 acre-feet.

Original designs included an 8,000 ft^3/s concrete overflow spillway to pass floodflows. During construction, slide movements on the unstable abutment above the spillway forced abandonment of the spillway, leaving only the 2,000 ft^3/s outlet works for reservoir discharge. It was recognized that additional discharge capability was needed and in 1961, an overflow spillway was added to the outlet structure. This increased reservoir discharge capacity to approximately 4,200 ft^3/s.

Using new procedures and updated hydrometeorological data, a new PMF for the watershed above the dam was developed. This flood would have a peak reservoir inflow of 32,700 ft^3/s and 5-day volume of 69,000 acre-feet (85 by 10^6 m^3). Routing the flood through the reservoir and outlets indicated that the existing dam could be overtopped by as much as 1.6 feet. Because this overtopping would cause dam failure, corrective measures were determined necessary [5]*.

ALTERNATIVES

The basic types of corrective measures considered to mitigate risks of property damage and loss of life due to dam failure include (1) revising reservoir operating procedures, (2) flood plain zoning for areas downstream from the dam, (3) breaching and abandoning the dam, (4) constructing upstream storage, and (5) modifying existing structures.

All nonstructural modification alternatives, including the no action alternative, were eliminated from further consideration because they either did not sufficiently reduce potential loss of life and property damages or they would result in excessive loss of project benefits. Constructing additional upstream storage was eliminated because of high costs. Modification of the existing structures was chosen as the least costly alternative since it would provide the necessary protection against dam failure without reducing project benefits.

Structural Modifications

Modifications to the existing structure to prevent overtopping caused by the PMF could be accomplished by increasing reservoir discharge capability, increasing reservoir storage capacity, or a combination of these. Preliminary designs and cost estimates resulted in the cost comparison shown in table 4-1.

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*Numbers in brackets refer to entries in the Bibliography.

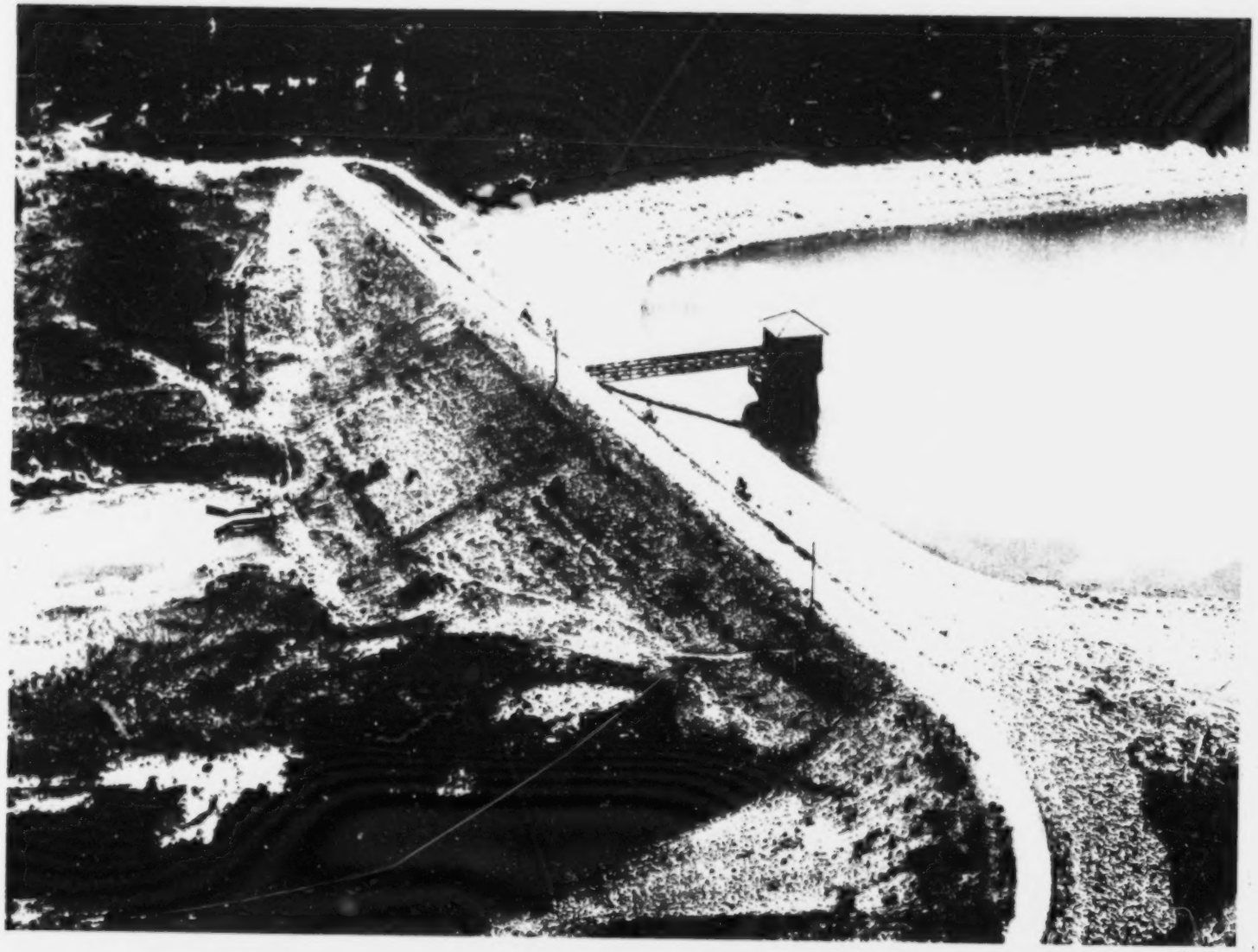

PLATE 6

Figure 4-1. - Lake Sherburne Dam before modification.

Table 4-1. - Cost for structural modifications

Modification	Construction costs ($ million - January 1981)
Raise crest of dam (retaining structure)	3.6
Raise crest of dam (embankment)	5.3
Construct auxiliary spillway	7.5
Construct auxiliary spillway and raise crest	5.8

Raising the dam's crest elevation would provide the capacity necessary to store flood inflow and allow discharge through the existing outlets. The method of raising the crest elevation usually includes adding earthfill over the existing embankment and extending the slopes to the desired crest elevation (fig. 4-2). This method is normally cost effective if adequate sources of construction materials are nearby and extensive modification to existing appurtenant structures is not necessary.

Raising Lake Sherburne Dam with embankment fill would require approximately 120,000 yd^3 of earthfill from sources as far as 8 miles from the dam. Modifying the dam's spillway-outlet works stilling structure would also be necessary. With the short summers in northern Montana, two construction seasons would be needed to complete the work. However, raising the dam with a retaining wall system (fig. 4-3) would require only 28,000 yd^3 of earthfill, no modification to the stilling structure, and only one season to construct.

Based on the significant cost advantage noted in table 1, raising the crest with a retaining wall system was selected as the most practical method of structural modification.

RETAINING WALL SYSTEM

The retaining walls constructed at Lake Sherburne Dam used a patented system known as Reinforced Earth. The system consists of precast concrete panels held in place by steel strips embedded in earth materials. Major components of the system are shown on figure 4-4. Several thousand Reinforced Earth structures have been constructed worldwide, however, none had been used for this particular application. The Reinforced Earth Company, Arlington, Virginia, provided the wall system design based on requirements specified by the Bureau of Reclamation.

DESIGN REQUIREMENTS

The basic requirement for the modification was to make the existing dam safe from overtopping and consequent failure caused by the PMF. To provide a safe and reliable structure, the modification would have to be designed for the

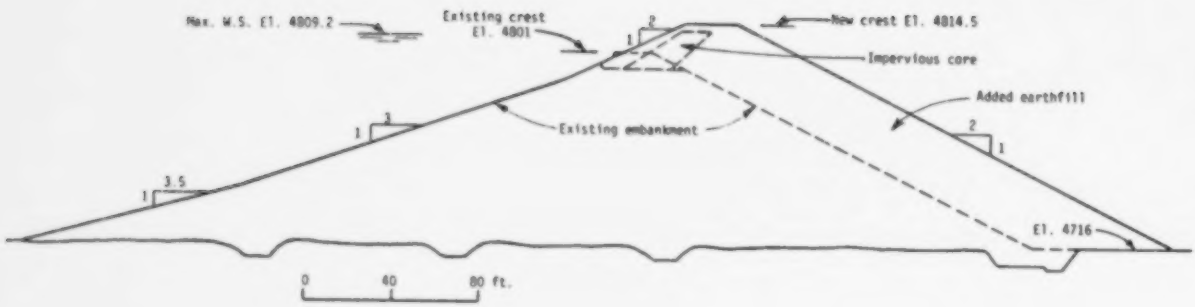

Figure 4-2. - Conventional method of raising dam crest elevation.

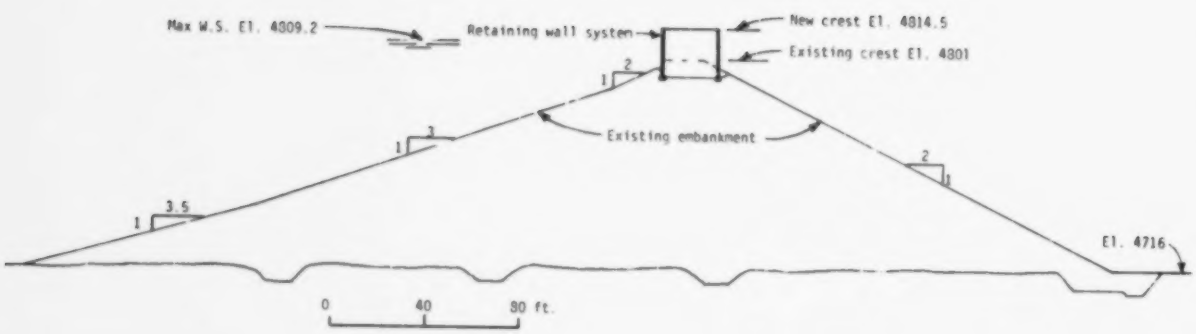

Figure 4-3. - Retaining wall method of raising dam crest elevation.

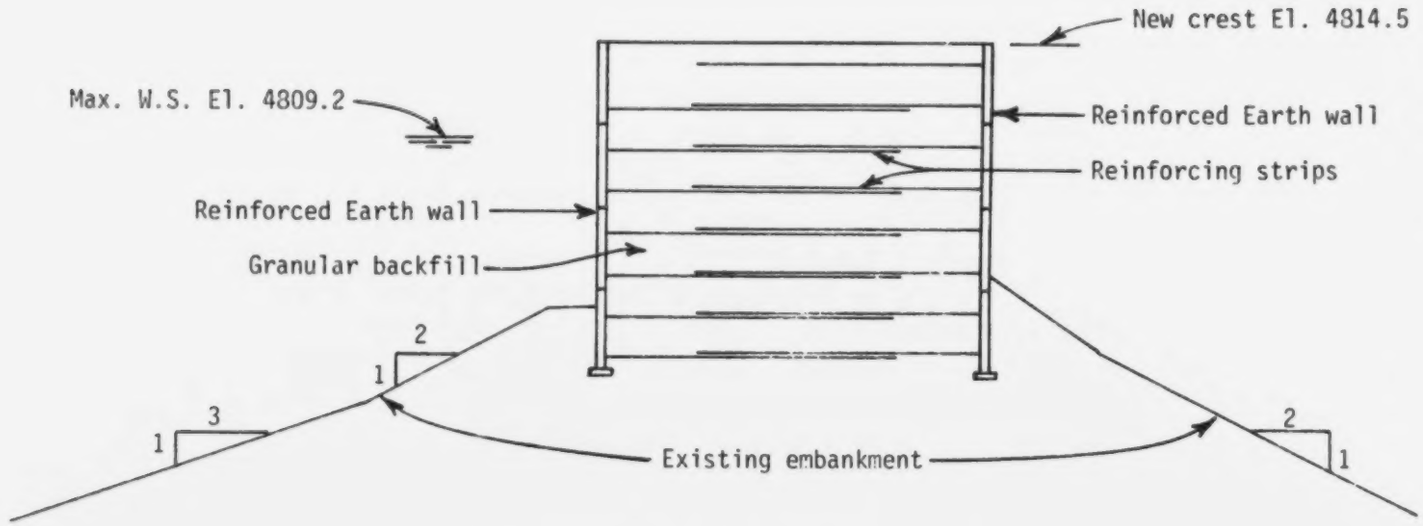

Figure 4-4. - Reinforced Earth retaining wall system.

unique site loading conditions imposed by the (1) design flood, (2) seepage forces, (3) static loads from the retaining wall system, and (4) dynamic loads from earthquakes. An additional design consideration, because of the modification method chosen, was the service life of the new structure.

Design Flood

Because of the potential hazards from dam failure, the PMF was selected as the design flood. Routing the design flood through the reservoir and outlets yielded a maximum water surface elevation of 4809.2 feet to provide sufficient surcharge space (41,950 acre-feet) and discharge capacity (4480 ft^3/s) to pass the design flood. Freeboard for wave runup and setup amounting to 5.3 feet was computed, yielding a new crest elevation of 4814.5 feet required to safely accommodate the flood event. The new crest elevation was about 14 feet higher than the existing dam crest. Therefore, after excavation of the upper 6 feet of embankment to provide an adequate foundation, the required height of the retaining walls was approximately 20 feet.

Static Loading

Conditions of static loading are those imposed by the additional weight on the embankment from the retaining structure and by the potentially higher reservoir water surfaces against the embankment and retaining structure. Evaluations of embankment stability for steady-state seepage and reservoir drawdown were necessary. Evaluation of overturning, base sliding, bearing capacity, and internal stability were required for the retaining wall system.

Stability computations for the nonmodified and modified embankment section under steady-state seepage and rapid reservoir drawdown were made using appropriate static stability methods [2]. The results of embankment static stability computations are shown in table 4-2. The retaining wall system was modeled as an external load applied to the embankment crest.

Table 4-2. - Results of embankment static stability analysis

Condition	Critical failure surface	Minimum safety factor
Nonmodified embankment		
Steady-state seepage	Downstream-circular	1.6
Rapid drawdown	Upstream-circular	1.2
Modified embankment		
Steady-state seepage	Downstream-circular	1.5
Rapid drawdown	Upstream-circular	1.2

These results show that the additional load from the retaining wall system and higher reservoir water surfaces had only minor effect on the embankment stability. The computed minimum factors of safety are within acceptable limits for these conditions.

Base sliding, overturning, and bearing capacity were considered with the retaining wall system acting as a gravity structure and full water load (maximum reservoir water surface) on the upstream face. The dimensions and weight of the structure, matched with the relatively small applied external load, yield high factors of safety against base sliding and overturning. Bearing capacity of the embankment materials was found to be satisfactory for the applied loads.

Internal stability for the Reinforced Earth system depends on the frictional resistance between the earth materials and the metal reinforcing strips. The backfill between the Reinforced Earth walls is retained by precast concrete panels, which are restrained by friction between the soil and the metal strips. The dimensions and density (number of strips per precast panel) of the strips are dependent on the wall height, external loading, and backfill soil properties.

Internal stability is determined by computing the active earth pressure and distribution on the walls, and thus the tensile forces on the reinforcing strips. Comparing these forces with the pullout capacity (frictional resistance) and tensile strength of the strips yields a factor of safety against failure of the wall. A simple computational procedure is used to determine the reinforcing strip density and dimensions required to obtain adequate factors of safety against strip pullout or failure in tension. As in the design for this modification, the pullout capacity of the reinforcing strips typically controls the design for walls up to about 20 feet high. The strip dimensions and density required for this design were controlled by dynamic loading conditions, which we discussed later.

Dynamic Loading

The MCE (maximum credible earthquake) for Lake Sherburne Dam has been set at an M (Richter magnitude) = 7.0 event at an epicentral distance of 20 miles and focal depth of 6 miles [10]. The February 9, 1971 earthquake near San Fernando, California, recorded in a Hollywood storage basement, is representative of an M = 6.6 event at 23 miles and focal depth of 8 miles, recorded over deep alluvium. The San Fernando earthquake record was chosen to represent the MCE. A scale factor of 1.3 times the accelerogram acceleration values was used to increase the earthquake intensity to an M = 7.0 event. The scaled acceleration time history for horizontal and vertical motion is shown on figures 4-5 and 4-6. The acceleration response spectrum is shown on figure 4-7.

It is necessary to determine the dynamic embankment response in evaluating expected embankment deformations and the performance of the Reinforced Earth structure during the MCE. The computer program SHAKE [8] was used to predict the embankment response. The usual procedure is to generate a rock motion from the ground surface motion, then to compute the response of the dam-foundation system. The depth to bedrock at Lake Sherburne is not well established, but has been set at 700 feet for the analysis. The effect of the embankment on the earthquake motion over deep alluvium would be small. For this analysis, the recorded ground motion was used as the motion at the base of the dam.

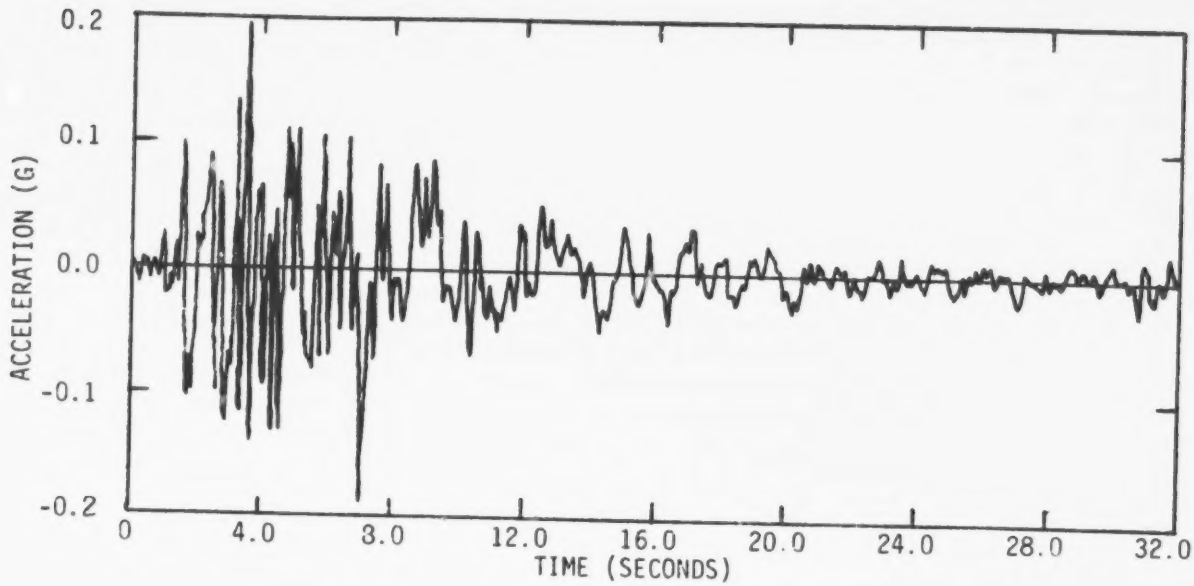

Figure 4-5. - Horizontal acceleration time history.

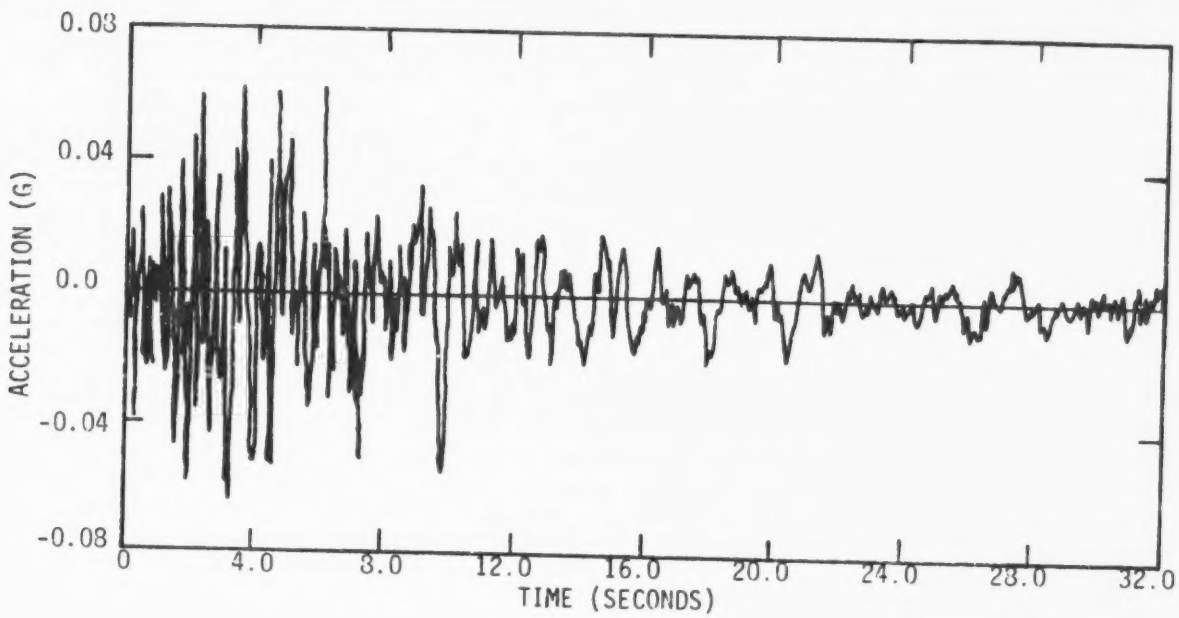

Figure 4-6. - Vertical acceleration time history.

Two methods were used to estimate the deformation of the dam under dynamic loading: a computer model [1] and a simplified analysis [9]. The simplified analysis requires the Amax (maximum crest acceleration) and fundamental period of the dam. These were calculated by the computer program SHAKE, using an estimated shear modulus for the Reinforced Earth structure and computed values of the shear modulus for the embankment. The shear modulus for the embankment was computed by the following relationship:

$$G = 1000\ k_2\ (\sigma m')^{1/2}\ lb/ft^2$$

in which G = the shear modulus, K_2 = the shear modulus parameter, and $\sigma m'$ = the mean effctive confining pressure. Values of K_2 were selected so that the computed values of shear modulus would bracket the embankment material for both dry and saturated conditions. The range of Amax and fundamental periods of the dam were computed as follows:

Case	Shear modulus parameter K_2	Fundamental period T_o	Maximum crest acceleration Amax
1	70, dry	0.70	0.32
2	70, saturated	0.75	0.32
3	120, dry	0.48	0.35
4	120, saturated	0.52	0.32

Case 3 has the fundamental period, T_o that is nearest the predominant period of the chosen earthquake, thereby producing the largest acceleration response. This case was used to estimate deformations by the simplified analysis. Case 3 was also used to develop the acceleration time history for postulated failure masses needed to compute displacements by the computer model.

The computer model (program DYNDISP) computes displacements, by double integration with respect to time, of the relative accelerations between the postulated sliding of the failure mass and its stable base. Relative displacements between the mass and sliding surface occur when the safety factor of the mass is less than one. The computed displacements for the downstream failure surface shown on figure 4-8, assuming no pore pressure development, is approximately 0.1 foot. No displacement calculations for an upstream failure are necessary since yield accelerations for the upstream failure surface are greater than those for downstream failure surfaces, assuming no dynamically induced pore pressures.

The simplified analysis to estimate deformations was done for comparison with the computer model. It was also used to calculate deformation for the rapid drawdown condition, assuming no pore pressure development. An implicit assumption of the simplified method is that no significant dynamically induced pore pressures have developed. This may or may not be valid, depending on the degree of saturation of the material and its behavior during shear. However, the approximate deformation calculated is adequate because the rapid drawdown case does not involve the loss of the reservoir. Maximum embankment deformations computed are:

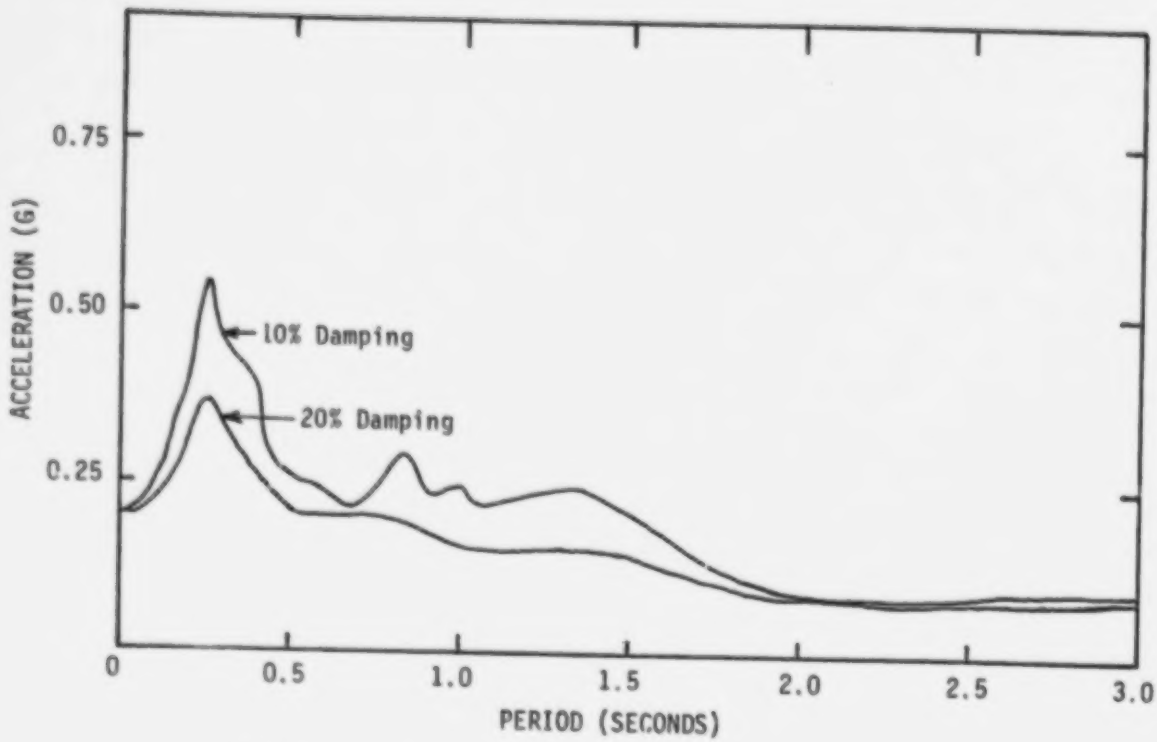

Figure 4-7. - Acceleration response spectrum.

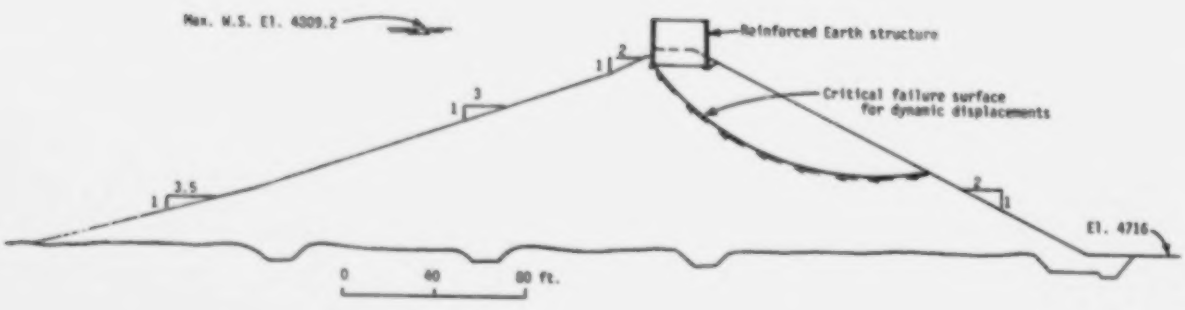

Figure 4-8. - Critical failure surface for dynamic displacements.

Steady-state seepage condition - 0.13 foot
Rapid drawdown condition - 0.64 foot

This amount of deformation is considered acceptable since it would not result in excessive loss of freeboard or significant damage to the embankment retaining wall system.

The internal stability of the Reinforced Earth system under earthquake loading conditions was analyzed by the Reinforced Earth Company according to procedures presented by Richardson [6]. These procedures determine the dynamic force magnitude in each reinforcing strip through computation of the amplitude and of the distribution of dynamic earth pressures acting on the walls. Sufficient reinforcement must be provided so that combined static and dynamic earth pressures do not cause tensile or soil-strip frictional failure of the reinforcement.

The steps in the design procedure are summarized as follows:

1. Selection of wall and reinforcing strip geometry and design parameters
2. Estimation of peak dynamic strains
3. Estimation of natural frequencies and damping
4. Development of design response spectra
5. Selection of spectral accelerations
6. Computation of total dynamic reinforcing strip forces based on amplitude and distribution of dynamic earth pressures

Design strip forces are obtained by combining the dynamic and static forces. Factors of safety are obtained by comparing the design forces with the ultimate tensile or soil-strip friction capacity of the strips. If changes in reinforcing strip density or wall geometry are necessary to obtain a desired factor of safety, the design procedure must be repeated since this will change the stiffness of the wall. The magnitude and distribution of the dynamic forces and peak dynamic strain are functions of the wall stiffness. An actual design acceleration time history for the site was used to determine an acceleration response spectrum at the base of the Reinforced Earth structure. This spectrum was obtained directly from the output of program SHAKE, thereby eliminating the need to develop a design response spectra by the more emperical method [6]. Using the actual design acceleration time history produced slightly higher spectral accelerations (0.45g versus 0.38g) than Richardson's method.

Dynamic stability against base sliding and overturning are checked by applying the horizontal acceleration and other loads to the structure.

Seepage

For the application proposed, the retaining wall system must provide a relatively watertight barrier and assurance that seepage would not affect stability. Flood routing studies showed that the reservoir water surface could be against the structure for as many as 8 days during the design flood event. Although water will be against the wall only during remote events, the structure must be capable of providing the necessary protection against seepage that could affect the integrity of the structure or embankment.

A viable means of providing a water barrier is to seal all joints on the upstream retaining structure wall. This could be accomplished by applying a waterproof membrane over the joints on the inside face of the wall, as shown on figure 4-9. Although the membrane should provide a positive water barrier, deformations of the wall during normal operations or unnoticed damage to the membrane during construction could disrupt its integrity. Assurance of a continuous membrane-to-wall seal would be difficult to control during construction. Therefore, as a defensive design measure, it was necessary to provide zoning of materials in the retaining wall system to further control any seepage not stopped by the membrane.

The zoning of materials should provide:

1. Sufficiently low permeability to restrict seepage flow to manageable quantities should disruption of the membrane occur.

2. Sufficiently high free-draining capability to prevent the retention of moisture around reinforcing strips, and thereby, minimize the potential for corrosion of the strips during normal operations.

3. Filter protection between adjacent zones to prevent the loss of material should seepage occur.

These requirements were met by designing the Reinforced Earth volume with the zoning shown on figure 4-10. Zone 2A, having the gradation limits of a silty sand, would act to restrict seepage flows. Courser gradations for zones 2 and 2B assure free-draining capabilities. Gradation of zone 2 was established from material available in the borrow. Gradation limits for zones 2A and 2B were designed to meet the requirements stated and applicable filter criteria.

Corrosion

An embankment dam with appurtenant structures is designed and constructed without major emphasis on service life because there is general concurrence that, with reasonable maintenance, it will last for decades. This has been verified by the number of dams built in the early 1900's and earlier that are still being operated safely. Experience, therefore, tells us that a service life of 100 years or longer is attainable, with few special provisions in the design. The safety of the reinforced Earth structure centers around embedding metal in earthfill materials and protecting that metal against the corrosion or loss of metal that could eventually compromise stability. It is

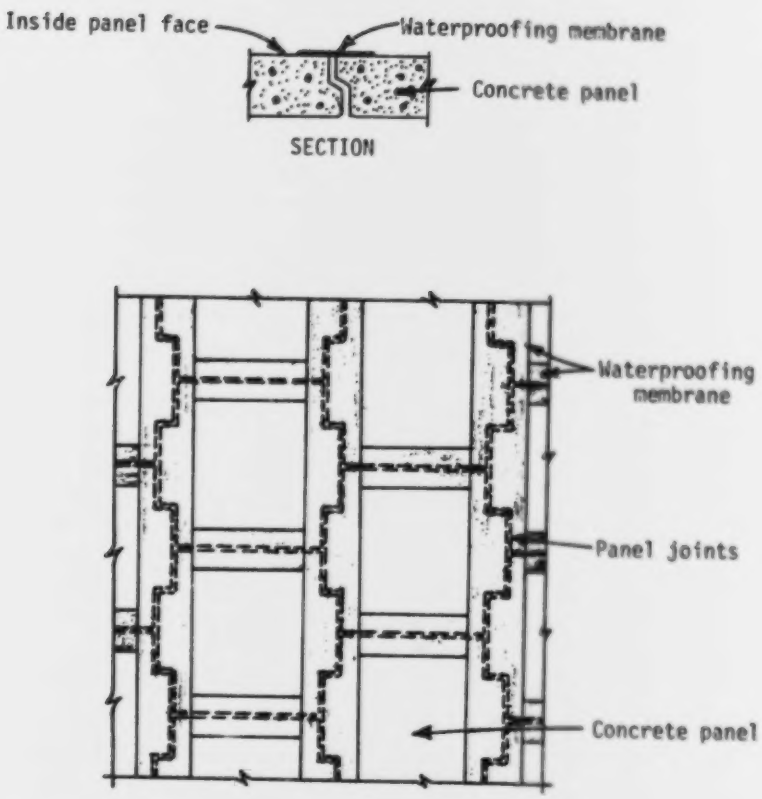

Figure 4-9. - Waterproofing membrane on wall joints.

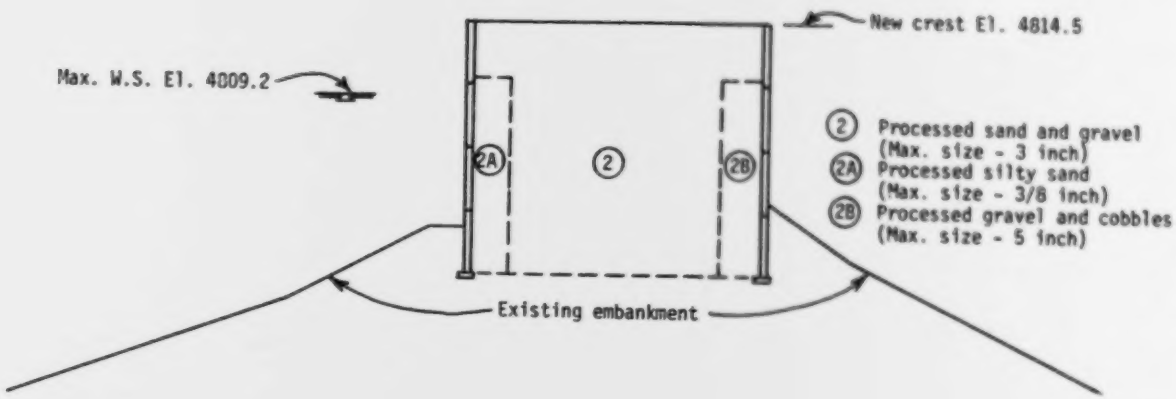
New crest El. 4814.5
Max. W.S. El. 4009.2
2A
2
2B
Existing embankment
2 Processed sand and gravel (Max. size - 3 inch)
2A Processed silty sand (Max. size - 3/8 inch)
2B Processed gravel and cobbles (Max. size - 5 inch)

Figure 4-10. - Zoning of earth materials between retaining walls.

highly desirable to design the Reinforced Earth structure with a service life comparable with that of the rest of the dam, so that it would not have to be replaced.

For typical applications, all embedded metalwork for Reinforced Earth structures are galvanized to provide protection against corrosion. Several thousand Reinforced Earth structures have been built worldwide since the first was completed in France, in 1966. Although there has been considerable experience with steel galvanized for corrosion protection, primarily with buried pipe, experience with galvanized steel in Reinforced Earth structures is limited, less than 20 years experience compared with the 50- to 100-year or longer service life desired. Several studies have been conducted to evaluate the long-term effectiveness of using galvanized steel to provide corrosion protection [3, 4, 7]. The studies have attempted to identify the environmental conditions affecting rates of corrosion.

As a result, current design practice for Reinforced Earth structures using galvanized steel includes provisions to control the environment surrounding the buried metal. These provisions include draining the fill and using nonaggressive soils, relatively free of clay and organic material. Nonaggressive soils are generally considered to be those with a pH between 6 and 9, and resistivity greater than 10,000 ohm-cm. Requiring these provisions to control the environment and providing a significant quantity of sacrificial metal are, therefore, the generally accepted design measures to assure durability under normal conditions. General conclusions and interpretations of various studies suggest service lives approaching 100 years using galvanized steel can be attained if: (1) the thickness of the metal is increased with additional sacrificial metal in marine environments or soils with high chloride content and (2) soils with high acid content and low resistivity are avoided [3]. When a more corrosive conditions exists and the environment of the embedment soil cannot be controlled economically, epoxy coatings may be used on the metalwork.

Although corrosion rates of buried metals have been studied fairly extensively, the corrosion potential produced by the embedment of metal in two dissimilar materials, soil and concrete, has not been addressed as extensively. However, it is known that a significantly higher potential for corrosion exists under these conditions, especially at or near the connections of the soil reinforcements to the concrete panel. Yet, the studies have generally considered metal loss from normal corrosion rather than the less understood mechanisms of local corrosion from pitting of the metal. Based on these observations, it was felt that better assurance of a design life comparable with that of the embankment was needed.

Several options to assure a better design life of the Reinforced Earth structure were considered for the modification. These options included:

1. Providing cathodic protection so that the rate of metal loss for the embedded metalwork could be effectively controlled and monitored.

2. Insulating the connection between the metal embedded in soil and the metal embedded in concrete to reduce the potential for corrosion.

3. Using an epoxy coating on all reinforcing components embedded in soil or concrete to reduce the potential for corrosion.

Cathodic protection was considered the most effective means of controlling corrosion; however, it is also the most expensive and difficult method to install. Providing an insulated connection would require the special manufacturing and field installation of an insulating device. Epoxy coating was chosen as the most viable option for corrosion protection since the coating could be factory applied at a cost only slightly higher than that for galvanizing. In the relatively benign exposure provided by this application, epoxy coating of reinforcing components would expand the structure's service life extensively with little added expense.

CONSTRUCTION

Construction for the modification began in July 1982, with excavation of the upper portion of the existing embankment. Panel placement began after completion of the excavation, drainage pipe installation, and placement of the leveling pads for the Reinforced Earth walls. Over 48,000 ft^2 of Reinforced Earth walls were constructed between early August and completion of the work in November 1982.

Construction of a Reinforced Earth wall consists of a relatively simple procedure, using standard construction equipment and methods. The first row of precast panels is set in place on the leveling pad, and backfill is spread and compacted behind the panels to the first level of reinforcing strips. The reinforcing strips are then attached to the panels, and additional backfill is spread and compacted over the strips to the next level. This sequence is repeated until the wall construction is complete. External bracing of the first row of panels is necessary; however, succeeding rows are held in place by clamping and wedging against the previous row.

A major consideration in choosing the retaining wall system was the time required for construction of the Reinforced Earth structure, less than 5 months, thereby allowing completion in one construction season. This greatly reduced the impact on project operations, which were necessarily restricted during portions of the construction. The more conventional method of raising the dam by adding embankment would have required two construction seasons, restricting reservoir operations during two irrigation seasons.

Total contract costs for the modification amounted to approximately $1.6 million, significantly less than original estimates. About 65 percent of the cost was for materials and construction associated with the Reinforced Earth structure.

SUMMARY

A safety evaluation of Lake Sherborne Dam resulted in the determination that modifications were necessary for protection against potential overtopping and consequent failure. The evaluation of alternatives for modification showed

that the most economical solution was to raise the dam's crest elevation, thereby providing the necessary storage capacity to allow safe passage of floodflows. The method chosen for raising the dam included a different application of a well-established retaining wall system known as Reinforced Earth. Different methods of analysis and design were used to account for unique site loading conditions, and thereby obtain a safe design. These design methods and considerations are recommended for consideration for other applications of this type.

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AUTOMATIC MONITORING OF STRUCTURAL BEHAVIOR INSTRUMENTS FOR RECLAMATION CONCRETE DAMS

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January 1984

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ABSTRACT

A microprocessor based system to record, process, and transmit structural behavior data from five concrete arch dams has been developed by the Bureau of Reclamation. Completion of this project also required the concurrent development of sensors to electrify readings previously taken visually from plumblines, weir levels, uplift pressure gages, and other instruments. Software has been developed on a desktop computer to initiate telephone transmission of data from the systems to the computer for further processing and archiving.

The major features of the monitoring system are:

1. Each of the five systems is composed of a master station and remote stations, distributed throughout the dam to reduce the amount of cabling between the sensors and the points where analog signals are converted to digital results.

2. A two-wire, high-speed data link is provided between each master station and its remote monitors.

3. The master station is provided with input/output capability for telephone communications, lake stage/tailwater elevation, and a Bureau of Reclamation developed plumbline monitoring system.

4. All master and remote units have completely interchangeable hardware and software.

5. Each system is designed to automatically make a telephone call to alert responsible personnel, if a predetermined limit is exceeded.

6. The system may be programmed and data may be acquired by telephone from a remote terminal or computer from anywhere in the world.

INTRODUCTION

Pierre Londe, honorary president of ICOLD (International Commission on Large Dams), recently developed a checklist to improve the safety of new earth dams during and after construction. Two of the ten points on this checklist concern instrumentation. They are:

1. Instrumentation is a vital part of dam design.
2. Thorough surveillance of dams is necessary, preferably on a 24-hour basis, together with automatic recording instrumentation.

The USBR (Bureau of Reclamation) has long recognized the importance of instrumentation in the design, construction, and operation of dams under its jurisdiction. During the last 5 years, an increased emphasis has been placed on the timely use of instrumentation data for surveillance and analysis of USBR structures. In 1978, reorganization of the Engineering and Research Center brought together instrumentation data gathering functions for concrete and embankment dams for the synergistic application of the latest computer technology. Since the new equipment still relied on manual input of hand-recorded data transmitted by mail, a program was developed to provide electronically gathered data, automatic transmittal of any out-of-limit warning values, and, on demand, a record of all the latest structural behavior data from USBR dams.

Although the dam monitoring automation program encompasses a wide range of areas, from landslide surveillance to embankment and concrete dam instrumentation throughout the western United States, it is the recent work on five concrete arch dams that this paper will address.

Monticello Dam, which is a feature of the Solano Project located 50 miles west of Sacramento, California, was selected as a field test site for the prototype instrumentation automation project. A larger monitoring project is now in progress and includes four dams on the CRSP (Colorado River Storage Project) (see fig. 5-1).

To be successful, an instrumentation program of this size and complexity requires support from all people involved, from top management personnel to the dam tender at the site. Funding support for this project was obtained from several sources. Development of the original plumbline monitor and installation of the industrial data logger and the developmental dam monitoring system at Monticello Dam were funded by PRESS (Program Related Scientific Studies) funds. The CRSP Power Operations Office provided funding for the installation of sensors and data monitoring systems at the four CRSP dams.

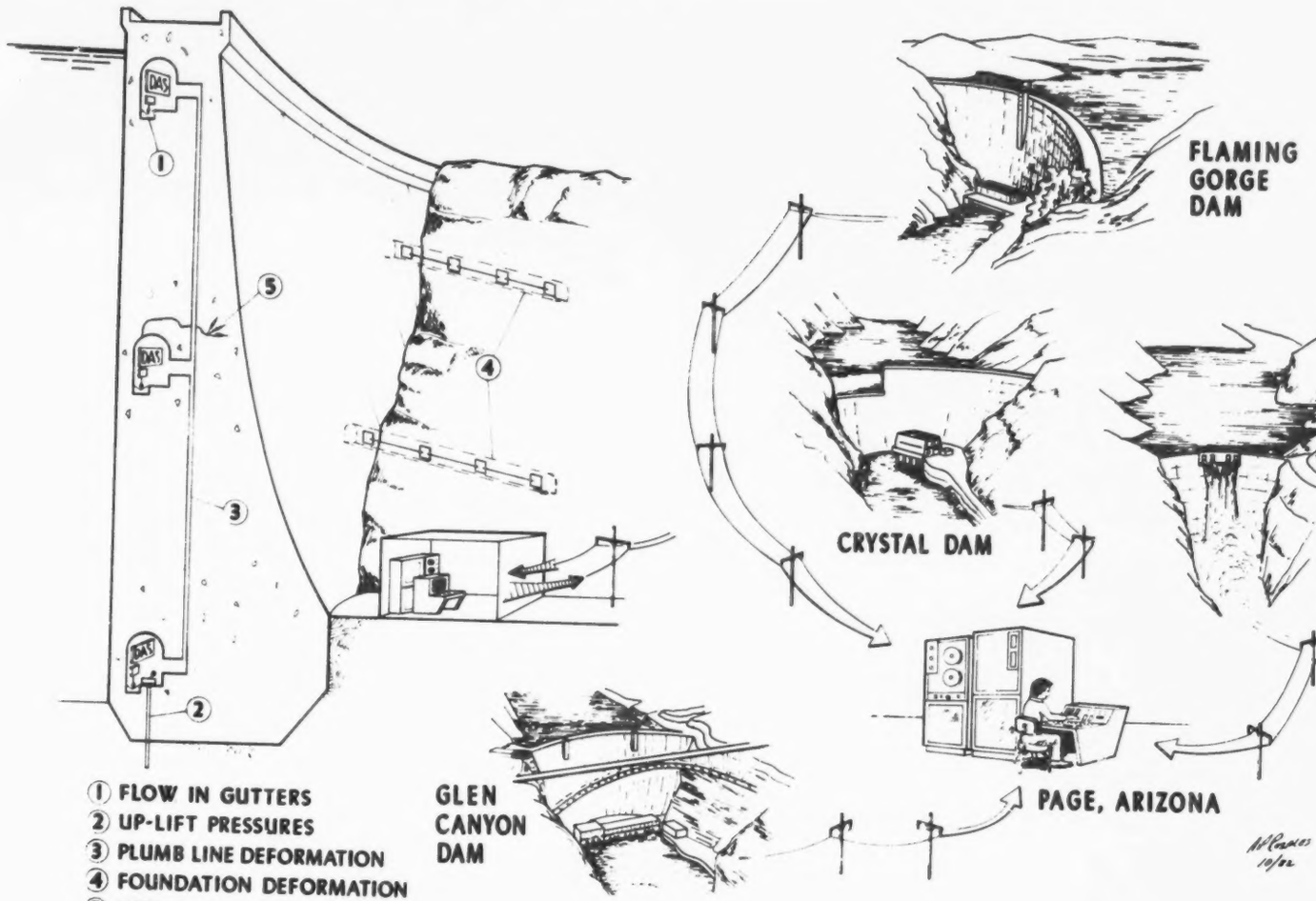

Figure 5-1. - Colorado River Storage Project, structural behavior instrumentation, dam monitoring automation. (The artists conception at left depicts typical sensors and monitoring instrumentation in a concrete dam. The rest of the drawing shows the relationship of the four Colorado River Storage Project dams to a central surveillance site.)

CONCLUSION

The development of an automatic system to monitor the behavior of a group of concrete dams appears to be a practical and effective means of efficiently collecting, transmitting, storing, processing, evaluating, and reporting structural behavior data to multiple locations with various needs. The integration of many technical, organizational, and personal needs required goal setting early in the project and the flexibility to make changes as the project progressed.

AUTOMATION GOALS

Early in the dam monitoring program, a diverse set of needs were identified. Engineers at the design office required timely presentation of structural behavior data in the form of plots, charts, printouts, etc., to ascertain whether the structures met design criteria. Regional and project personnel also needed this information to meet their operation and maintenance responsibilities. In addition, personnel at the dams saw the need for local displays of selected instrument readings for inspection purposes.

To meet these diverse requirements, a system was designed which would provide for the distribution of sensor data within each structure. Data monitors were strategically located throughout the dam and linked to a master station which would gather the information for transmission outside the dam. The system was required to be operator-friendly to allow menu-driven (question and reply) programming by local personnel. A terminal was supplied at the master station to allow the operator to monitor data, program sensors, and perform diagnostics throughout the system. Each data monitor has a miniterminal attached, which allows the operator to perform local scanning and calibration of sensors without returning to the master terminal.

Telephone transmission of data from the master station at the dam to outside locations was selected as the most universal means of data distribution. Microcomputers interfaced to the telephone system at the regional and design offices can perform data request, receipt, storage, and presentation functions.

TRANSDUCER SELECTION

To establish compatiblity with the data monitors, all of the transducers must have an electrical output. Previously, the embedded Carlson stress, strain, and temperature meters were manually read and recorded using a portable electrical resistance measuring instrument. Resistance measuring ports and data reduction software are provided in the new data monitors to indicate strain and temperature data from these meters.

Seepage at various locations in each dam is presently monitored by measuring the distance from a wall-mounted index marker to the water surface behind V-notch and rectangular weirs in gallery gutters. This

measurement is reproduced electrically by submerging a depth measurement pressure transducer upstream from the weir and monitoring the voltage. This voltage is proportional to the water head. Using scaling and offset factors stored in the monitor, the distance from the index mark to the water is reproduced for manual checking. After data transmission, this distance is used by the microcomputer to calculate the flow using the weir formula.

Foundation uplift pressures are measured by connecting a pressure transducer to the same pipe that supplies a manually read pressure gage. The monitor scales the transducer output voltage to obtain the pressure value.

A foundation deformation measurement system is used at several of the dams on the CRSP. Tunnels up to 300 feet long, driven along the horizontal extension of the dam's axis, contain a series of Invar surveying tapes. These horizontally positioned tapes are suspended at intermediate points, fixed at one end and free to move at the other. Parallel mounted surveying springs provide constant tension to the tape through a yoke at the free end. A manual measurement of the tunnel deformation is made by a depth micrometer, which measures the distance between a wall mounted angle fixture and a measuring button on the tension yoke. To electrify the deformation measurement, a new tensioning fixture was designed to replace the undependable tension springs and to provide an attachment for a LVDT (linear variable differential transformer) displacement transducer. The transducer is shown on figure 5-2, attached to a fulcrum and constant-weight tension fixture.

The sensors described above comprised the major portion of the project's instrumentation and the availability of the remote monitors in the dam allows easy integration of new instruments. A comparison study of tiltmeter and plumbline instrumentation to be conducted at Morrow Point Dam is an example of this integration.

SENSOR DEVELOPMENT

While most of the sensors used to monitor a dam's structural behavior parameters can be purchased off-the-shelf, sensors for electrically converting certain readings were not readily available. For example, the concrete dam's plumbline monitor required a development program to devise a system which would automatically gather wire position data and transmit it to the dam monitor master station when requested.

Plumblines have historically been used in most of the USBR large concrete dams to detect upstream-to-downstream and abutment-to-abutment deflections. These deflections are caused by many sources including thermal effects, reservoir loading, creep, and foundation settlement. During construction, plumbline wells are cast vertically through the dam and are designed to intersect with transverse galleries. At these intersections, blockouts are formed where manual reading instruments are installed to measure the orthogonal movement of the reading station in relation to the suspension point of the plumbline. This manual reading method requires

an operator to visually align a screw-driven sighting device with the plumbline wire and then record, on a printed form, a numerical value representing the position of the wire, taken from a vernier scale or micrometer barrel. The readings, taken in two perpendicular axes, define the wire's location in a horizontal plane.

The plumbline readings orbit through a somewhat predictable yearly cycle. These readings relate mainly to the structure's temperature and secondarily to the reservoir water surface. Most of the other contributing parameters, like concrete and foundation creep, change slowly with time. Should any of these parameters change, there would be a discontinuity in the historical, temperature induced orbit of the plumbline wire.

The first USBR automatic plumbline monitor consisted of a servo-controlled following device that incorporated two perpendicularly mounted screw-driven actuators using noncontacting displacement feedback transducers. This mechanical system was abandoned because of difficulty in finding a suitable manufacturer and because a system with no moving parts was considered to be more reliable in the adverse environment found in a dam.

A second approach to develop a plumbline monitor was taken using a new technology incorporated in industrial automatic vision systems. These systems use solid state electronic components to project the image of an object illuminated by a light source. In 1982, the Bureau entered into a contract with Spectron Engineering, Inc., of Denver, Colorado, to design, produce, and install a prototype automatic plumbline monitoring system at Monticello Dam, California. The system contained three X-Y cameras, a microcomputer, and a printer. It's components, shown on figure 5-3, were required to measure the wire's position over a 3-inch range with a resolution of 0.0001-inch in both X and Y axes. The camera system was also required to fit inside a 10-inch-diameter plumbline well without interfering with the originally installed manual reading equipment.

The plumbline monitor system senses the position of the wire and converts that position into two sets of numerical values corresponding with the X-Y axes of the sensor. The first calculations made reproduce the readings obtained from the manual peepsight device by the dam operator. These values provide a quick visual comparison of the results from the two systems and instill operator confidence in the automatic systems readings. The second set of values calculated by the system are the radial and tangential positions of the wire referenced to the geometry of the dam. When taken manually these values are not produced on site, but are computed later using data reduction programs at the Bureaus's central computer system in Denver. The microcomputer was programmed to scan, compute, and print these values and the peepsight readings, either at the operator's request or automatically every 24 hours. Data plots of automatically and manually obtained readings taken from the center block at Monticello Dam during a 7-month period are shown on figure 5-4.

After the evaluation of the prototype plumbline monitor, a second contract was awarded to produce four systems containing 26 sensors to be installed at the four CRSP dams. The contract schedule required

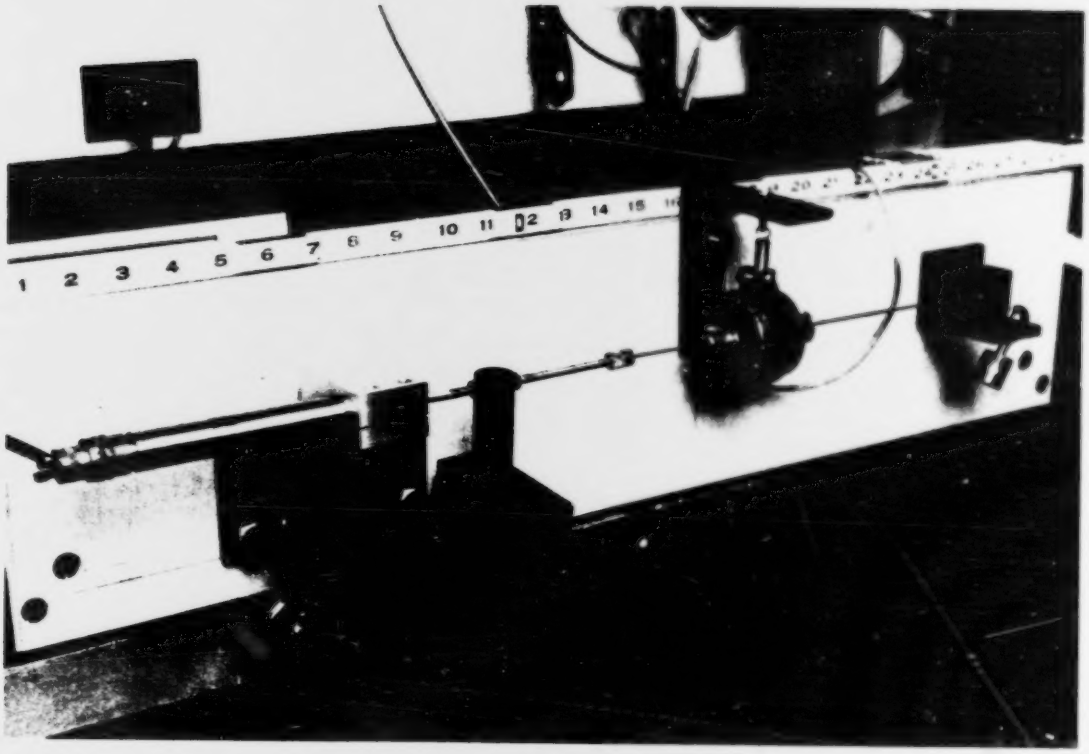

Figure 5-2. - Foundation deformation instrumentation in a laboratory test stand.

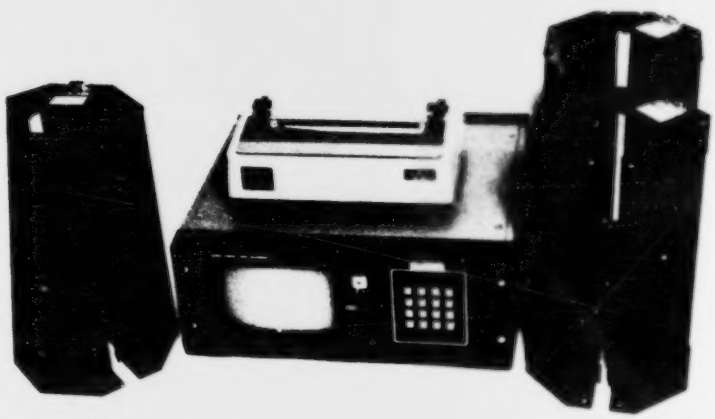

Figure 5-3. - Plumbline monitoring system prior to installation at Monticello Dam, California. The three uphole sensors are scanned by the microcomputer, then the data is reduced and printed or transmitted to another computer.

MONTICELLO DAM
AUTOMATIC AND MANUAL PLUMBLINE READINGS

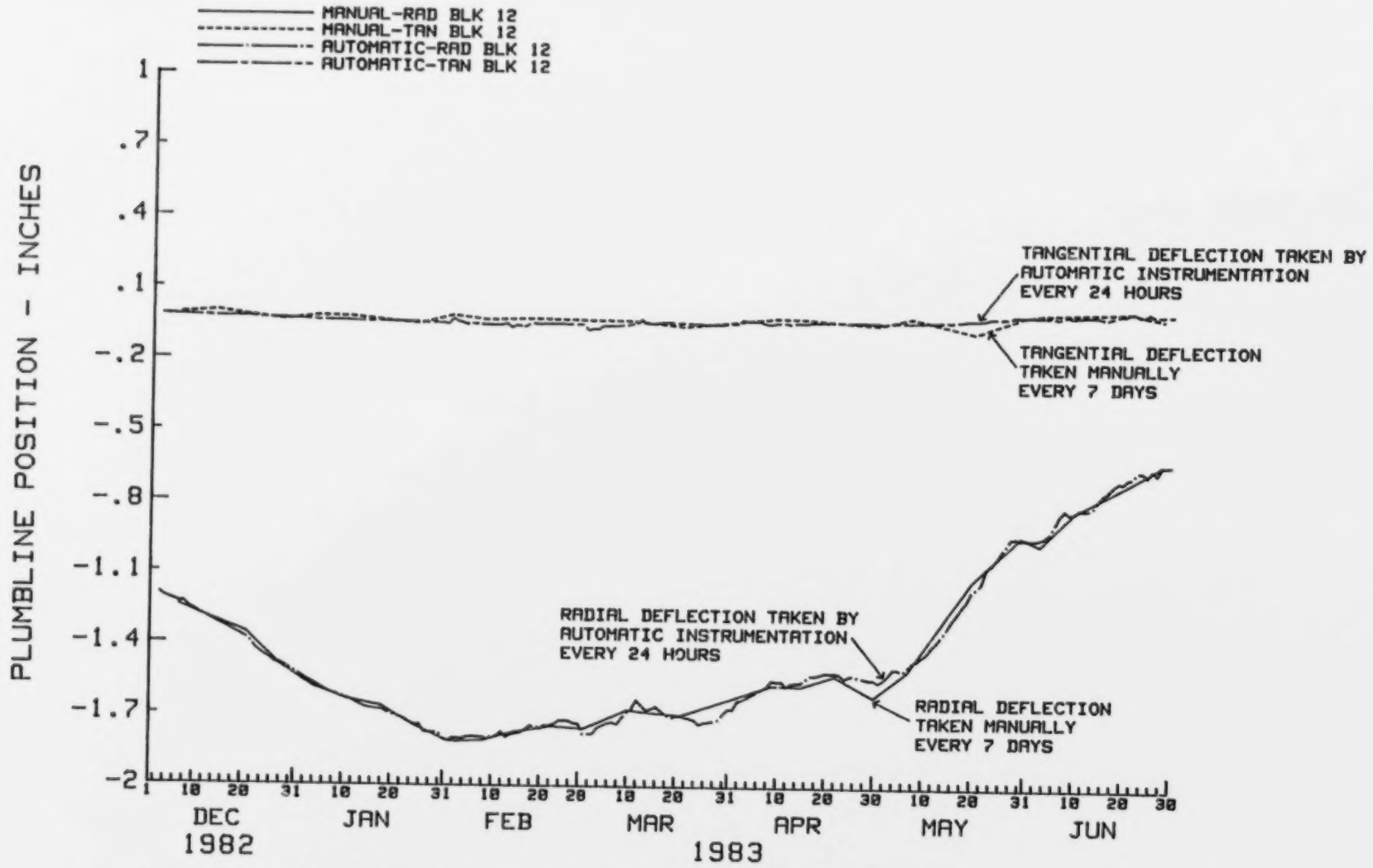

Figure 5-4. - Comparison of radial and tangential plumbline readings taken automatically and manually in the center block of Monticello Dam, California.

completion by December 1983. In addition to the Monticello Dam design, the CRSP contract included a requirement for graphical display of the plumbline position and the installation of the hardware and software needed for two-way communication between the master dam monitor and the plumbline monitor's microcomputer.

AUTOMATIC DAM MONITOR DEVELOPMENT

The first of the recent projects for the automatic recording of structural behavior instrumentation was performed in 1981, at Monticello Dam. One hundred fifty-eight channels of Carlson strain and temperature meters were read and recorded every 6 hours using a standard industrial type data logger (manufactured by Acurex Corporation of Mountain View, California) and a magnetic tape recorder (see fig. 5-5). The data tapes were mailed every 2 weeks to the Engineering and Research Center in Denver for processing on the central computer system.

Several things were learned from this first installation and operation of a centralized system that later influenced the design of the distributed dam monitor. The major items were:

- One of the major expense items for the project was cabling. Cable lengths to provide power to each sensor and carry the analog signal back to the digitizer for recording should be kept to a minimum. From this experience a system of smaller distributed monitors was envisioned.

- A readout device located near the sensor installation would decrease the time and effort required to install and check out the sensor.

- Each data point should be accompanied by identification information describing its function and location.

- Data from specifically developed microprocessor based instrumentation, such as the plumbline monitor, can be collected at the dam monitor through a digital interface. These data can then be tagged with time information from the system clock and transmitted in the same block of data containing other sensor scans.

- Each data set needs to be identified with location, time, and project designators.

- Each measured electrical signal should be converted into engineering values and units at the time it is scanned.

- The data should be accessible upon demand by an operator, and the system should be programmable at places both inside and outside the dam.

- Security must be provided against unauthorized or accidental intrusion into the system.

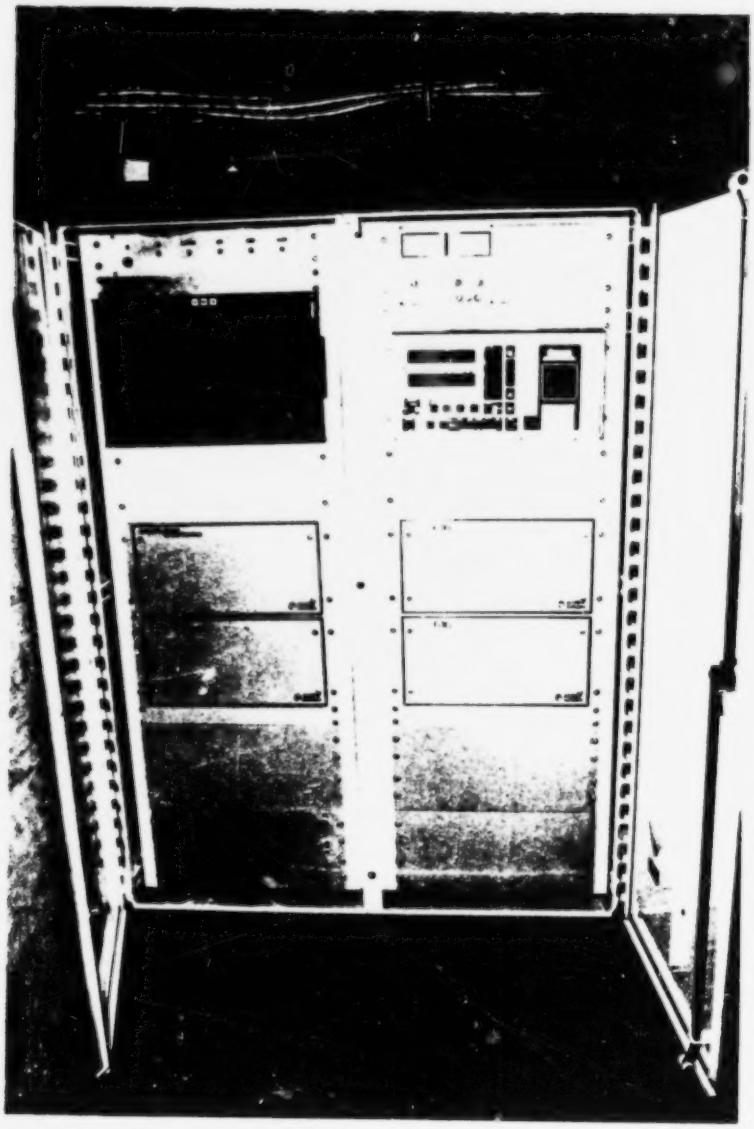

Figure 5-5. - An industrial data logger system installed at Monticello Dam, California. The moisture proof cabinet contains the data logger, tape recorder, power supply for sensor excitation, and an error enunciator panel.

- Provisions should be made to test the memory and communications of each monitor.

- User programming of each parameter should be in a question and answer dialog.

- Alarm functions should be provided to alert personnel at the dam and at several other locations of possible structural problems in the dam.

- The cabinet depth should not exceed 1 foot to allow for passage in the 5-foot-wide gallery.

During the Spring of 1982, information gathered during field trips to the four CRSP dams, the experience with the centralized system at Monticello Dam, and the study of data acquisition system manufacturers literature led us to believe that a custom or tailored system was required to produce the best monitoring of the concrete dams. With this in mind, a contract was awarded in October 1982 to Foundation Science, Inc. of Portland, Oregon, for the manufacture and installation of five automatic dam monitoring systems.

The distribution of the dam monitors on the CRSP dams was as follows:

- The master station was located in the powerplant near the telephone service and the operator's console.

- A remote monitor was located at the toe of the dam, where many of the uplift pressures and seepage flows are measured.

- Remote monitors (see fig. 5-6) were located at two positions near each abutment, to take data from strain meters, seepage flow sensors, and foundation deformation instruments, except at Crystal Dam where only one monitor was located at each abutment.

The master station and each of the remote monitors have interchangeable hardware and software. Simply by changing address switches, any remote monitor can be changed into the master station and vice versa. Accessories provided by the master station, and not by the remote monitors, are: a real-time system clock, the plumbline monitor interface card, lakestage/tailwater input, a telephone modem, an extra 64K byte random access memory card for data storage, and a control card for communications between the master and the remote units.

A block diagram of the dam monitoring function is shown on figure 5-7. The STD bus backplane was selected as the basis for the system to standardize the architecture and provide for the greatest selection of function cards for future expansion. This bus standard also has the greatest number of manufacturing sources for specific function cards.

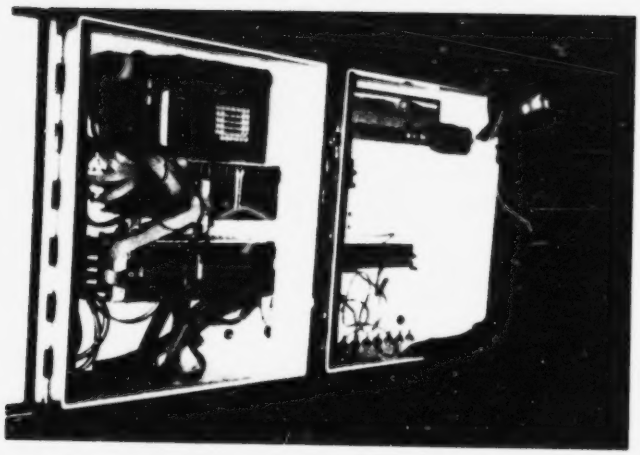

Figure 5-6. - Distributed data monitor shown mounted in a gallery at Flaming Gorge Dam, Utah. Computer, analog, and digital processors, and the units mini terminal are in the left cabinet. The right portion contains sensor excitation and input connections.

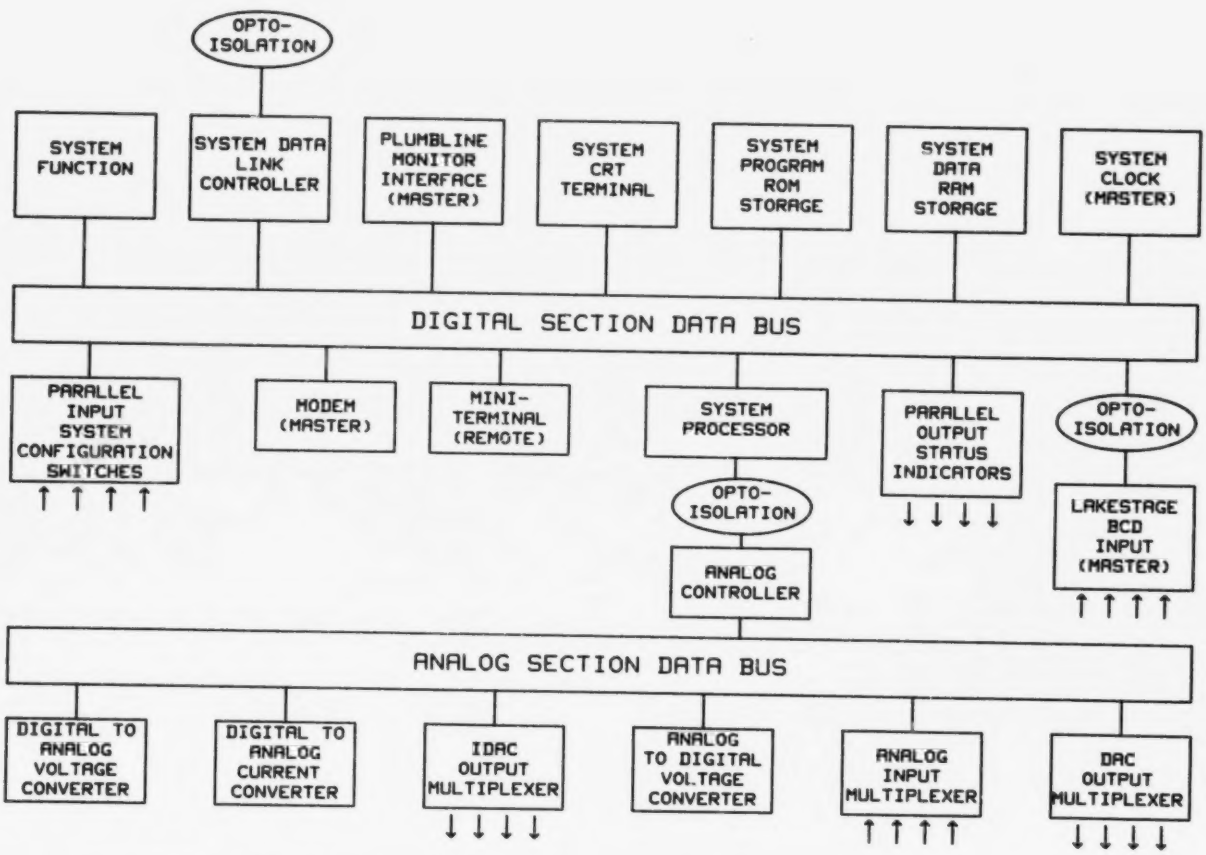

Figure 5-7. - Block diagram of the dam monitor hardware functions.

Another consideration in the dam monitoring system specifications was protection from electrical transients. These fluctuations have various origins, but come mainly from lightning, generator service interruptions, and power system grid breakdowns. Protection is provided by three means. First, the a-c power supplied to each monitor goes through a high-isolation transformer for noise filtering and spike suppression. Second, the data link, lakestage input, and analog controller are optically isolated from the digital bus. By converting digital electrical signals to light pulses and back to electrical signals, the path for high voltage signals is broken. Finally, each sensor input contains sparkgap lightning protection.

To produce a dam monitoring system that will be the most versatile for the widest variety of personnel and with the least training effort, a menu-driven monitor program and diagnostic software package are supplied with the system. A flow chart showing the software functions is shown on figure 5-8. In the idle state the monitoring system is able to perform any of the programmed scanning and alarm functions as well as accept keyboard entries. Remote entry requires a low level security code to prevent unauthorized access.

In the terminal entry state, a general directory allows either data monitoring, system programming, system diagnostics, or return to the idle state. When not used by the dam personnel, the system is left in the idle state to service remote terminal access and sensor scans.

The data monitoring portion of the software allows display of scaled transducer data, units, and alarm annotation. Another data display is available with an additional 20-character description feature. This description can provide location data and other information about the transducer. Selection of lakestage/tailwater elevations and values transmitted from the plumbline monitor may also be selected. A display, called "postcard," is available for showing a free field, 80 column by 20 line text, that is entered from either the remote or local terminal, in the programming section. This feature allows the operator to add notes of explanation or information concerning pertinent visual observations. A high level security code is not required for remote entry into the data monitoring section of the software.

The program directory section of the software allows the terminal operator to change the date/time, scan and plumbline reading intervals, and dam/project designation, as well as enter new "postcard" information. Setup of each channel is performed in the channel parameters portion of this section. To set up a channel the instrument type (voltage, thermocouple, resistance, etc.), units, calibration offset and scale factor, alarm levels, and 20-character descriptive message are entered. The channel parameter subsection also allows for the programming of an excitation voltage for sensors such as strain gages. Entry into the program directory section from a remote terminal requires a high level security code.

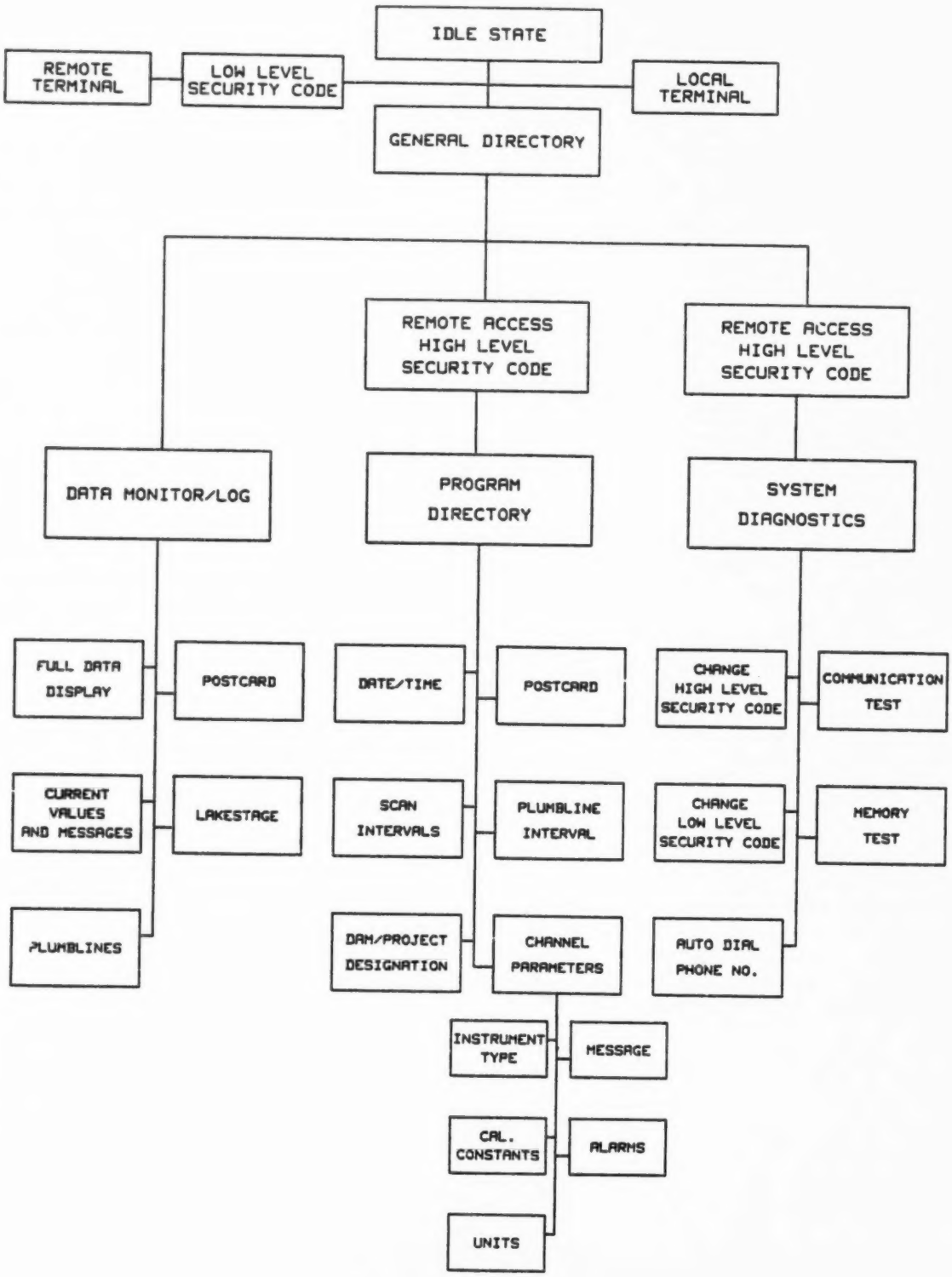

Figure 5-8. - Dam monitor software flow chart.

System diagnostics also require a high level remote access security code. Its function is to provide for changing security codes and automatic dial telephone numbers, and for performing memory and inter-system communications testing.

During the development of the dam monitoring system, several changes were implemented. These changes resulted from the concerns of field personnel and project visits, during which necessary revisions were foreseen. The flexibility that this microprocessor based system gives to the user is nearly unlimited. Changes in hardware or software to perform other functions can be made easily. Future plans for the monitor include daily hard copy data printouts, a local desktop computer with software to produce reports and graphs, and other enhancements to meet the needs of the system's users.

IMPROVEMENTS TO USBR PINHOLE TEST EQUIPMENT DESIGN AND TEST RESULT EVALUATION

by

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January 1984

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Engineering and Research Center
Bureau of Reclamation

Denver, Colorado

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ABSTRACT

A revised design for the USBR (Bureau of Reclamation) pinhole test apparatus has been developed and evaluated at the Engineering and Research Center and two regional project laboratories. The equipment design modifications have simplified the specimen placement procedures, placement density determinations, conducting the test, and interpreting the test results. Project laboratories performed 407 pinhole and crumb tests for USBR project feature investigation and construction control. The flow rate data from these pinhole tests have been evaluated and used to refine the original pinhole test dispersive grade versus flow rate criteria.

INTRODUCTION

The pinhole test was originally developed during the early 1970's, by Dr. James L. Sherard to provide qualitative information on soil erosion resistance to flowing water. Until that time, it had been recognized that some clayey soils naturally and readily deflocculate, or disperse, in the presence of water. The physico-chemical characteristics of the colloidal clay particle and water system were recognized as the primary variables involved in soil dispersion. However, quantitative chemical test methods were known to have limitations as standard laboratory tests for identifying dispersive soils. The work of early researchers investigating soil dispersion [1, 2, 3]* resulted in simple qualitative methods for indirectly assessing soil dispersion. The indirect test methods of Volk and Emerson, later popularized as the double hydrometer test and the crumb test, respectively, were well-known to soil scientists and agricultural engineers, but were relatively unknown to geotechnical and civil engineers. Sherard used these tests to corroborate pinhole tests and soil pore-water chemistry data on soils with erosion characteristics that were known from field performance. In 1976, Sherard's pinhole test [4] became the first qualitative physical test where the soil's erosion resistance to flowing water was directly observed during the test.

SUMMARY AND CONCLUSIONS

The results of 407 pinhole and crumb tests are presented in summary form in the appendix. The data were obtained as the result of the first USBR field evaluations conducted for the identification, classification, and construction control of dispersive soils. USBR project laboratories at the McGee Creek damsite in Farris, Oklahoma, and the Central Arizona Project in Phoenix, Arizona, were provided equipment, and personnel were trained to conduct and interpret pinhole, crumb, and double hydrometer tests. Liaison between the field project laboratories and the E&R (Engineering and Research) Center laboratory staff ensured that field testing and data analysis resulted in correct identification and classification of dispersive and nondispersive soils. All field test data show that the pinhole and crumb tests can be conducted easily and reliably at construction project laboratories when proper training and followup data analyses are provided.

Figure 6-1 shows criteria for dispersive grade classification versus flow rate and was developed from several hundred individual pinhole tests. The classification boundary zones provide a 95 percent confidence level for correctly classifying soil dispersive grades using pinhole test flow rate data.

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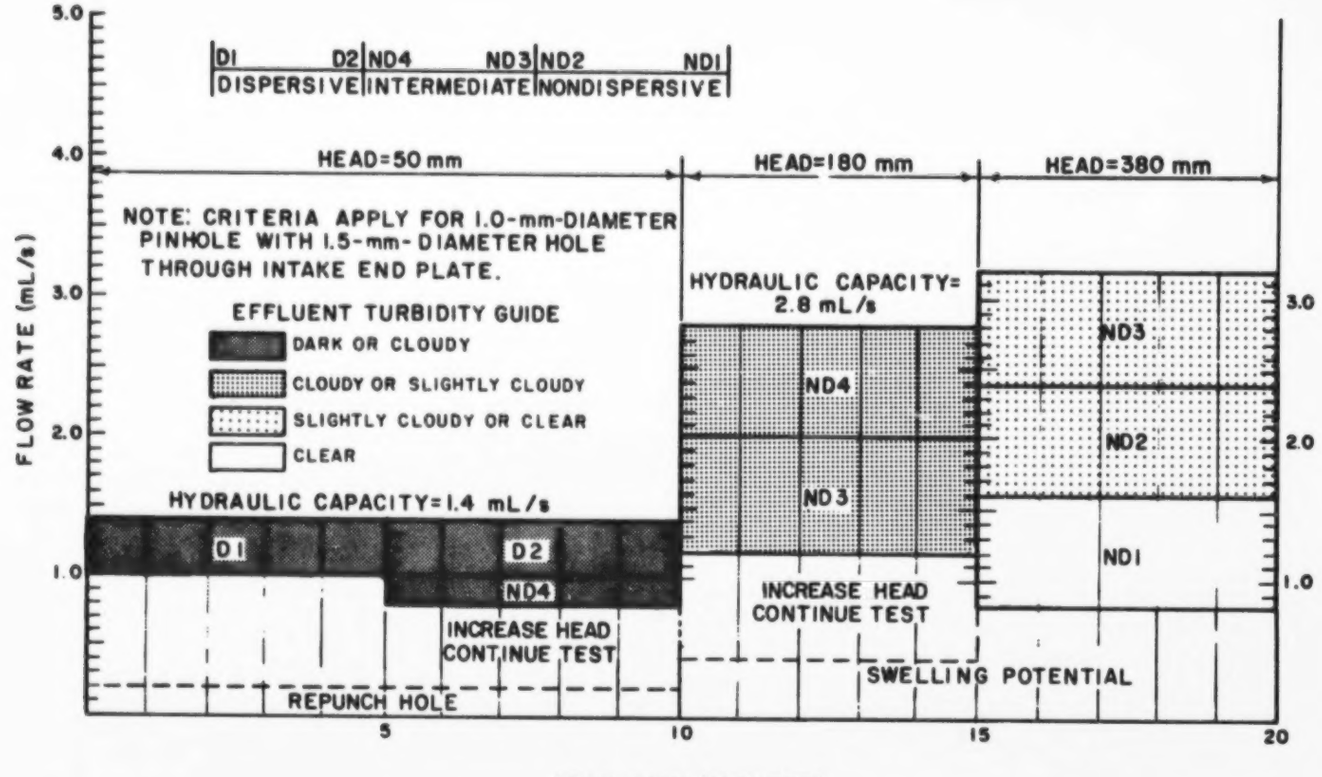
* Numbers in brackets refer to entries in the Bibliography.



DISPERSIVE GRADE VS FLOW RATE

SAMPLE NUMBER	PLACEMENT CONDITIONS		DISPERSIVE GRADE CLASSIFICATION
	DRY DENSITY	WATER CONTENT	

Figure 6-1. - Pinhole test dispersive grade versus flow rate.

DEVELOPMENT OF ORIGINAL PINHOLE TEST APPARATUS

Concurrent with Sherard's pinhole test investigations in the early 1970's, Dr. Haliburton of Oklahoma State University, in cooperation with the USBR, developed the physical erosion test. The physical erosion test equipment, test procedure, and soil test results were documented in 1975 [5]. During the period from 1975 to 1977, the original USBR pinhole test apparatus was developed. Since both the pinhole and the physical erosion test were new state-of-the-art soil dispersion identification tests, the original USBR pinhole test apparatus was designed to be compatible with the physical erosion test apparatus. The pinhole specimen cylinders and cylinder end plates were identical to those used in the physical erosion test. Special pinhole specimen packing plugs were designed to account for the shorter length pinhole specimen. Figure 6-2 shows the original USBR pinhole test apparatus.

From 1975 to 1977, both the pinhole and physical erosion tests were evaluated using soils from miscellaneous USBR project features. This trial evaluation resulted in adopting the pinhole test as the preferred USBR laboratory test for identifying dispersive soils.

The development of the original USBR pinhole test apparatus and test procedure was based on Sherard's paper [4] and documented in 1979, by James L. Kinney [6]. A significant difference between the USBR and the Sherard pinhole apparatus was the use of a 2.2-mm-diameter hole in the truncated cone centering guide (nipple) of the USBR apparatus. The Sherard pinhole apparatus has a 1.5-mm hole. Appendix A of Kinney's report [6] explains why the hole in the nipple was enlarged to 2.2 mm to achieve the system flow rates published in Sherard's paper [4].

The USBR pinhole apparatus with the enlarged nipple hole was used from March 1977 until July 1981. During this period, several problems were experienced using the original USBR design of the pinhole apparatus [6] and Sherard's pinhole test evaluation criteria [4]:

1. The minimum hydraulic cross-section of the USBR pinhole apparatus (2.2-mm diameter) was significantly greater than Sherard's pinhole apparatus (1.5-mm diameter), resulting in different flow rates at the same hydraulic head [7].

2. The effluent turbidity criteria were difficult to interpret and introduced operator bias in arriving at final dispersive grade classifications [7].

3. Evaluation of measured flow rates appeared to be a reasonable, technically sound approach for arriving at dispersive grade classifications, but preliminary investigations showed problems (appendix A of [6]).

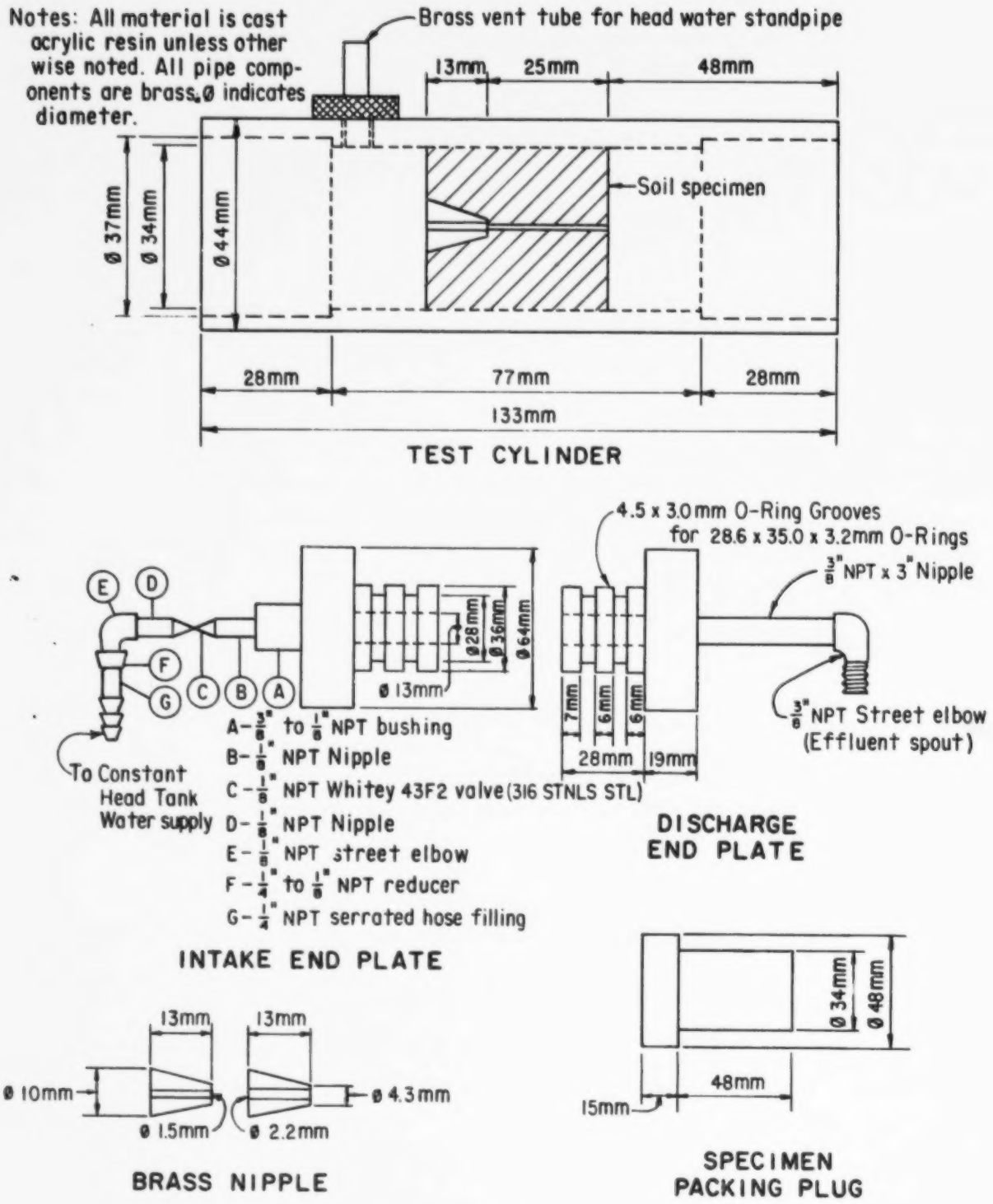


Figure 6-2. - USBR pinhole test apparatus - original design.

The problem areas identified above were resolved as the result of extensive investigations conducted from July 1981 through January 1982. Several significant results of that investigation are fully documented in reference [7] and briefly summarized below:

1. Equipment design problems were identified with Sherard's original pinhole apparatus and caused the actual hydraulic head on the test specimen to be greater than the nominal hydraulic head.

2. When Sherard's flow rate data were adjusted to correct the actual and nominal hydraulic head, it was found that flow rate data could be used to accurately reclassify dispersive soils that had originally been classified on the basis of effluent turbidity.

3. Based on 1 and 2 above, the use of the 2.2-mm-diameter nipple hole was discontinued in July 1981, and the 1.5-mm-diameter nipple hole was reinstated for use in the USBR pinhole apparatus.

4. Dispersive grade versus flow rate criteria were developed using Sherard's original data and routine pinhole tests conducted at the USBR E&R Center laboratories. The pinhole test dispersive grade versus flow rate criteria were originally published as figure 15 of reference [7].

FIELD EVALUATION OF USBR PINHOLE TEST APPARATUS

In February 1980, a dispersive clay field testing program was discussed and formalized for material invesigation and construction control of project features of McGee Creek Dam, near Farris, Oklahoma. The details of that testing program are contained in reference [8]. The pinhole test apparatus shown on figure 6-2 was initially used at the field project laboratory. The procedures outlined in reference [6] and supplemental information contained in reference [8] were used by field laboratory personnel for conducting pinhole and crumb tests.

References [9, 10, 11] document the results of visits to the McGee Creek project laboratory made from March 1981 to May 1982. During the field evaluation of the original pinhole test apparatus, several operating difficulties were identified:

1. The length of the test specimen cylinder, in relation to its diameter and the location of the soil specimen in the cylinder, hindered access from each open end. The specimen location made it difficult to properly position and insert the nipple centering guide for punching the pinhole.

2. The specimen compaction procedure required placing a known mass of moist soil to a predetermined final lift thickness and compacting using impact-type compaction. Determining final placement density was difficult because surface irregularities in the final compacted surface affected the average specimen length measurement.

3. The specimen conditioning procedure required a minimum of 16 hours curing in the test cylinder after compaction. This requirement was

originally specified in reference [6] to ensure uniform moisture distribution in the test specimen before conducting the test. This overnight curing requirement posed a problem for field construction control where the construction schedule would require daily decisions based on pinhole test results.

4. The glass beads used upstream and downstream of the soil specimen were awkward to work with and required special handling when the pinhole specimen needed to be repunched before or during the test.

5. Inserting the nipple with the 1.5-mm-diameter hole caused local densification of the test specimen in the area immediately adjacent to and downstream of the nipple. There was no way to quantify the effect of the local densification resulting from inserting the nipple, however, eliminating the nipple would simplify specimen handling procedures.

Figure 6-3 shows the USBR pinhole test apparatus with design modifications that were made to resolve the problem areas noted during the field trial evaluation of the original pinhole test apparatus. The new pinhole test apparatus produces the same results as the original design with the following advantages:

1. The pinhole specimen mold contains only the cylindrical soil specimen required for the test. This eliminates the problem of limited access to the specimen and allows the final placement density to be determined directly using conventional measurement techniques.

2. The specimen compaction mold was designed to be compatible with commercially available Harvard miniature compaction equipment, which uses a spring-loaded tamper. Specimens compacted with the spring-loaded tamper are more uniform than those prepared using impact type compaction.

3. The nipple centering guide with the 1.5-mm-diameter hole has been eliminated. The 1.5-mm-diameter hole has been incorporated in the equipment intake end plate.

4. The equipment end plates are connected directly to the upstream and downstream faces of the specimen. This eliminates the use of glass beads and greatly reduces the effort required to disassemble the apparatus to repunch the specimen pinhole.

5. The requirement to cure compacted specimens 16 hours before testing has been eliminated. The 16-hour curing requirement was originally specified to allow dissipation and equalization of excess pore pressures, resulting from the compaction process, before performing the test. It was originally thought that the dispersion of clay colloids might be adversely affected by excess pore water pressures. However, further analysis indicated that the physico-chemical forces responsible for dispersion were considerably greater than the interparticle forces resulting from the compaction process. The uniform distribution of moisture throughout the specimen is ensured by allowing the specimen to mellow in the loose state before compaction. The test is conducted immediately after compacting the specimen.

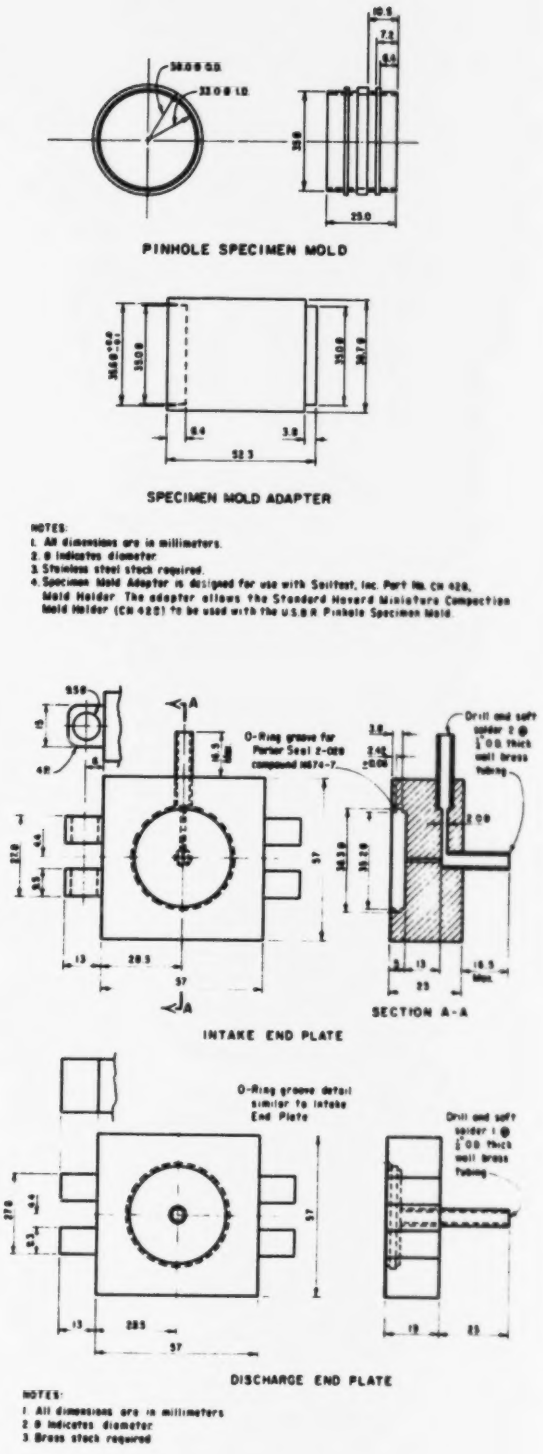
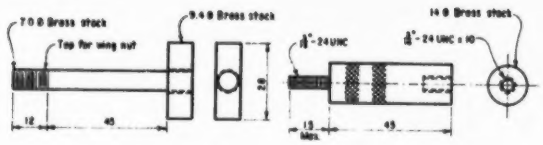
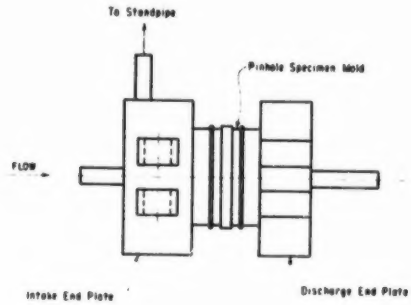

PINHOLE SPECIMEN MOLD
SPECIMEN MOLD ADAPTER
NOTES:
1. All dimensions are in millimeters.
2. Ø indicates diameter.
3. Stainless steel stock required.
4. Specimen Mold Adapter is designed for use with Soiltest, Inc. Part No. CN 420, Mold Holder. The adapter allows the Standard Hevard Miniature Compaction Mold Holder (CN 422) to be used with the U.S.B.R. Pinhole Specimen Mold.
O-Ring groove for Parker Seal 2-026 compound N674-7
Drill and soft solder 2 Ø 1/8" O.D. thick wall brass tubing
SECTION A-A
INTAKE END PLATE
O-Ring groove detail similar to Intake End Plate
Drill and soft solder 1 Ø 1/8" O.D. thick wall brass tubing
DISCHARGE END PLATE
NOTES:
1. All dimensions are in millimeters
2. Ø indicates diameter
3. Brass stock required

Figure 6-3. - USBR pinhole test apparatus - modified design (sheet 1 of 2).

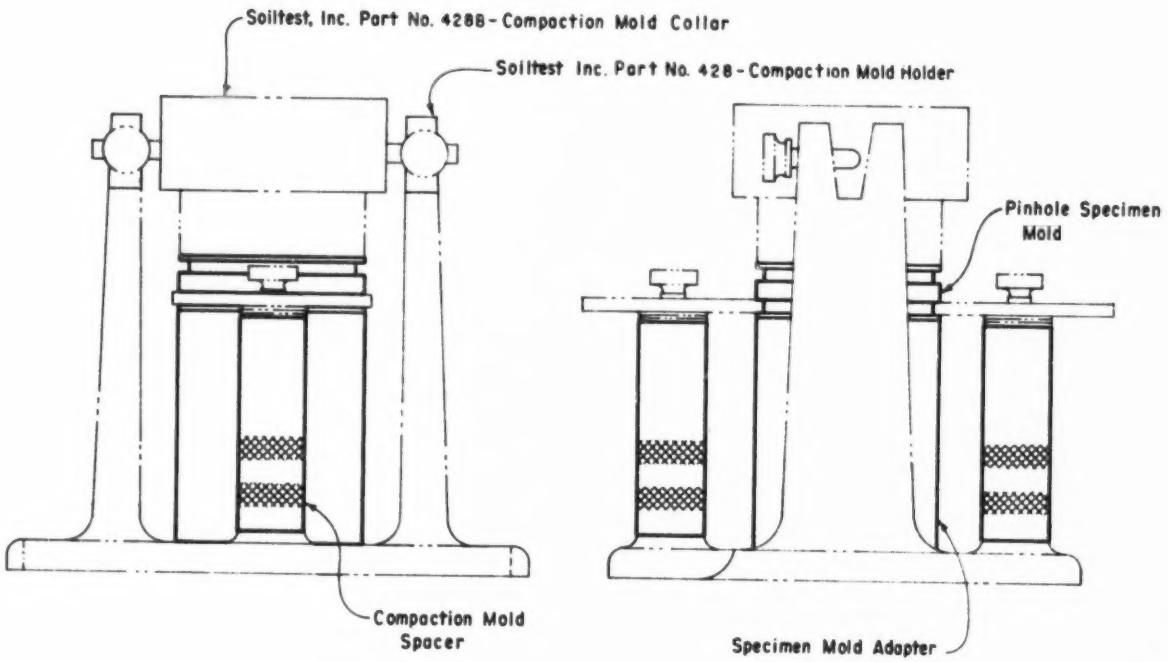


WING NUT & BOLT ASSEMBLY
(END PLATE CLAMPING DEVICE)

COMPACTION MOLD SPACER



PINHOLE TEST APPARATUS - ASSEMBLY DRAWING
(WING NUT & BOLT ASSEMBLY OMITTED FOR CLARITY)


Soiltest, Inc. Part No. 4288 - Compaction Mold Collar
Soiltest Inc. Part No. 428 - Compaction Mold Holder
Pinhole Specimen Mold
Compaction Mold Spacer
Specimen Mold Adapter

PINHOLE SPECIMEN COMPACTION APPARATUS - ASSEMBLY DRAWING
(COMMERCIALLY AVAILABLE ITEMS SHOWN IN PHANTOM LINES)

Figure 6-3. - USBR pinhole test apparatus - modified design (sheet 2 of 2).

Field evaluation of the modified design pinhole test apparatus began at the McGee Creek Project laboratory in June 1982. Another field laboratory evaluation program began at the Central Arizona Project laboratory at Phoenix, Arizona, in May 1983.

RESULTS OF FIELD LABORATORY PINHOLE AND CRUMB TESTS

Appendixes A through J contain detailed summary pinhole and crumb test results conducted on 407 soil specimens selected from various project features at the McGee Creek damsite and the Central Arizona Project. The pinhole and crumb tests were conducted using the detailed test procedures described in appendixes K and L, respectively. The dispersive grade classifications and correlation of the pinhole and crumb test results are given in table 6-1 and on figure 6-4.

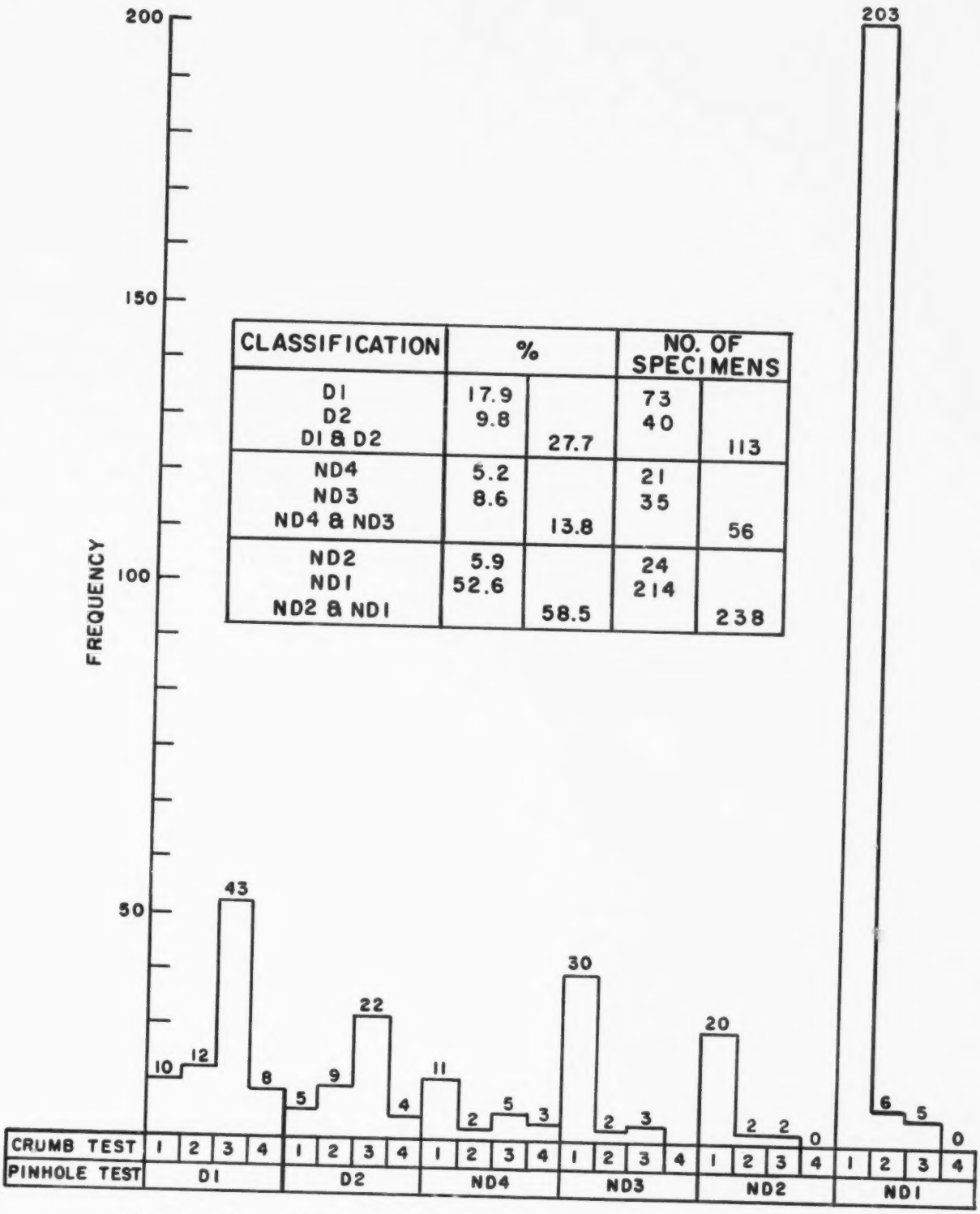
Table 6-2 shows the pinhole test dispersive grade classifications corroborated with the crumb test dispersive grade classifications. Dispersive grades from both the pinhole and crumb test results are categorized individually and in groups. The 407 individual data sets are assumed to be a representative data base because all field test data received are included. The distribution of dispersive (27.7 percent), intermediate (13.8 percent), and nondispersive (58.5 percent) specimens, as classified by the pinhole test, and dispersive (23.3 percent) and nondispersive (76.7 percent) specimens, as classified by the crumb test, are also considered representative.

The data shown in table 6-2 confirm several important observations of pinhole and crumb test correlations that previously have been reported in the literature [4, 12, 13, 14]:

1. In the pinhole test, the majority of specimens showed either dispersive or nondispersive reactions; intermediate reactions seldom occurred (56 out of 407 cases).

2. The crumb test classifies dispersive soils with greater reliability than it classifies nondispersive soils. Of the soils tested, 31 percent showed a grade 1 or 2 (nondispersive) crumb test reaction along with a D1 or D2 (dispersive) reaction in the pinhole test. Only 4.2 percent of the crumb tests showed grades 3 or 4 dispersive reaction with the pinhole test showing ND2 or ND1 nondispersive reactions (10 out of 238 cases). This confirms that a soil has a high probability of being dispersive if the crumb test indicates dispersion.

3. If ND4 soils (intermediate) are included with D1 and D2 dispersive soils, and the ND3 soils (intermediate) are included with ND2 and ND1 soils, the crumb tests' correlations are not significantly changed. Of the crumb tests, 36 percent showed grade 1 or 2 (nondispersive reactions) with the pinhole test showing dispersive reactions. Three percent of the crumb tests showed grade 3 or 4 (dispersive) reactions with the pinhole test showing nondispersive reactions. This confirms that a nondispersive reaction in the crumb test does not reliably indicate that the soil is nondispersive.



CLASSIFICATION	%		NO. OF SPECIMENS	
D1	17.9		73	
D2	9.8		40	
D1 & D2		27.7		113
ND4	5.2		21	
ND3	8.6		35	
ND4 & ND3		13.8		56
ND2	5.9		24	
ND1	52.6		214	
ND2 & ND1		58.5		238

Figure 6-4. - Dispersive grade versus frequency of occurrence.

Table 6-1. - Summary of dispersive grades (field test data)

Project feature and data source		D1				D2				ND4				ND3				ND2				ND1		
		1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3	4	1	2	3
Red Rocks Pumping Plant (app. 1)	23	4	-	2	-	-	1	1	-	-	-	-	-	4	1	-	-	-	-	-	-	10	-	-
Brady Pumping Plant (app. 2)	15	-	-	2	1	-	-	3	1	-	-	2	1	-	-	1	-	-	-	-	-	3	1	-
Tucson Aqueduct - R2 (app. 3)	56	-	1	1	-	3	3	-	-	5	-	-	-	5	-	-	-	1	-	-	-	33	4	-
Tucson Aqueduct - R3 (app. 4)	30	2	2	2	-	-	-	1	-	-	1	-	-	3	-	1	-	3	-	-	-	15	-	-
Cliff Dam D/S (app. 5)	14	1	-	1	-	-	1	1	-	-	-	-	-	7	-	-	-	-	-	-	-	3	-	-
Tucson Aqueduct - R1 (app. 6)	47	-	4	23	3	-	-	8	-	2	-	-	-	1	-	-	-	-	-	-	-	6	-	-
Tucson Aqueduct - R1 Borrow Area (app. 7)	68	3	5	10	-	2	4	4	1	2	-	-	-	6	1	-	-	1	-	-	-	27	1	-
Misc. specimens (- No. 10 and 4) (app. 8)	3	1	-	-	-	-	-	-	-	1	1	-	-	-	-	-	-	-	-	-	-	-	-	-
Lime treated soil McGee Creek Borrow Area F (app. 9)	147	-	-	2	4	-	-	4	2	1	-	2	2	4	-	1	-	14	2	1	-	103	-	5
Lime treated - stockpiled soil (app. 10)	4	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	1	-	-	-	3	-	-
All data	407	10	12	43	8	5	9	22	4	11	2	5	3	30	2	3	-	20	2	2	-	203	6	5

Table 6-2. - Pinhole test and crumb test data corroboration

Pinhole test		Crumb test results (total percent specimens)							
Class	Number of specimens	1	2	3	4	1,2	3,4	1	2,3,4
D1	73	14	16	59	11	30	70	14	86
D2	40	12	22	55	11	34	66	12	88
ND4	21	53	10	24	13	63	37	53	47
D1,D2	113	13	18	58	11	31	69	13	87
D1,D2, ND4	134	19	17	52	12	36	64	19	81
ND3	35	86	5.7	8.3	0	92	8	86	14
ND2	24	83	8.5	8.5	0	92	8	83	17
ND1	214	95	2.8	2.2	0	98	3	95	5
ND1,ND2	238	44	3.4	2.6	0	97	3	94	6
ND1,ND2 ND3	273	93	3.7	3.3	0	97	3	93	7
D1,D2 ND4,ND3	169	33	15	43	9	48	52	33	67
ND4,ND3 ND2,ND1	294	90	4	5	1	94	6	90	10
ND4,ND3	56	73	7.1	14	5.9	80	20	73	27

EVALUATION OF PINHOLE TEST FLOW RATE DATA

Appendix M contains a detailed summary of all 273 ND3, ND2, and ND1 pinhole test flow rate data at heads of 50, 180, and 380 mm. As previously stated in the literature [4, 6, 7, 12], for nondispersive soils, the original pinhole diameter remains unchanged and the flow rate remains constant at each head. For essentially all pinhole tests summarized in appendix M, the flow rates at each head remained constant for the entire duration of flow. The data entries represent the initial and final flow rates measured at each head. Appendix M is organized so that the frequency of occurrence of selected flow rate ranges could be easily determined. The data of appendix M are also shown in the histogram of figure 6-5. Figure 6-5 graphically shows the distribution of all flow rate measurements and the variation that can exist in any set of flow rate measurements.

Figure 15 of reference [7], was developed to relate Sherard's original pinhole test dispersive grades to flow rates obtained during the test. The major area of emphasis of the investigation for preparing reference [7] was the system flow rates for dispersive soils. For dispersive soils, the system flow rate is initially controlled by the 1.0-mm pinhole diameter, but is ultimately controlled by the hydraulic capacity of the 1.5-mm-diameter hole after the pinhole erodes. For nondispersive soils, the flow rate is always controlled by the pinhole, which remains 1.0-mm diameter throughout the test. Figure 15 of reference [7] was prepared from the pinhole test flow rate data of approximately 300 dispersive soil specimens and 60 nondispersive soil specimens. When the flow rate data of appendix M and figure 6-5 are compared with the flow rate boundary zones on figure 15 of reference [7], it is apparent that minor adjustments of the ND3 and ND1 lower flow rate boundaries are required.

Figure 6-1 shows the revised USBR pinhole test dispersive grade versus flow rate test evaluation criteria. Figure 6-1 was developed directly from figure 15 of reference [7], with revisions reflecting the flow rate data from the 273 nondispersive soil specimens in this report. Figure 6-1 shows dispersive grade classification boundary zones, which now give a 95 percent confidence level for correctly classifying soils using pinhole test flow rate data.

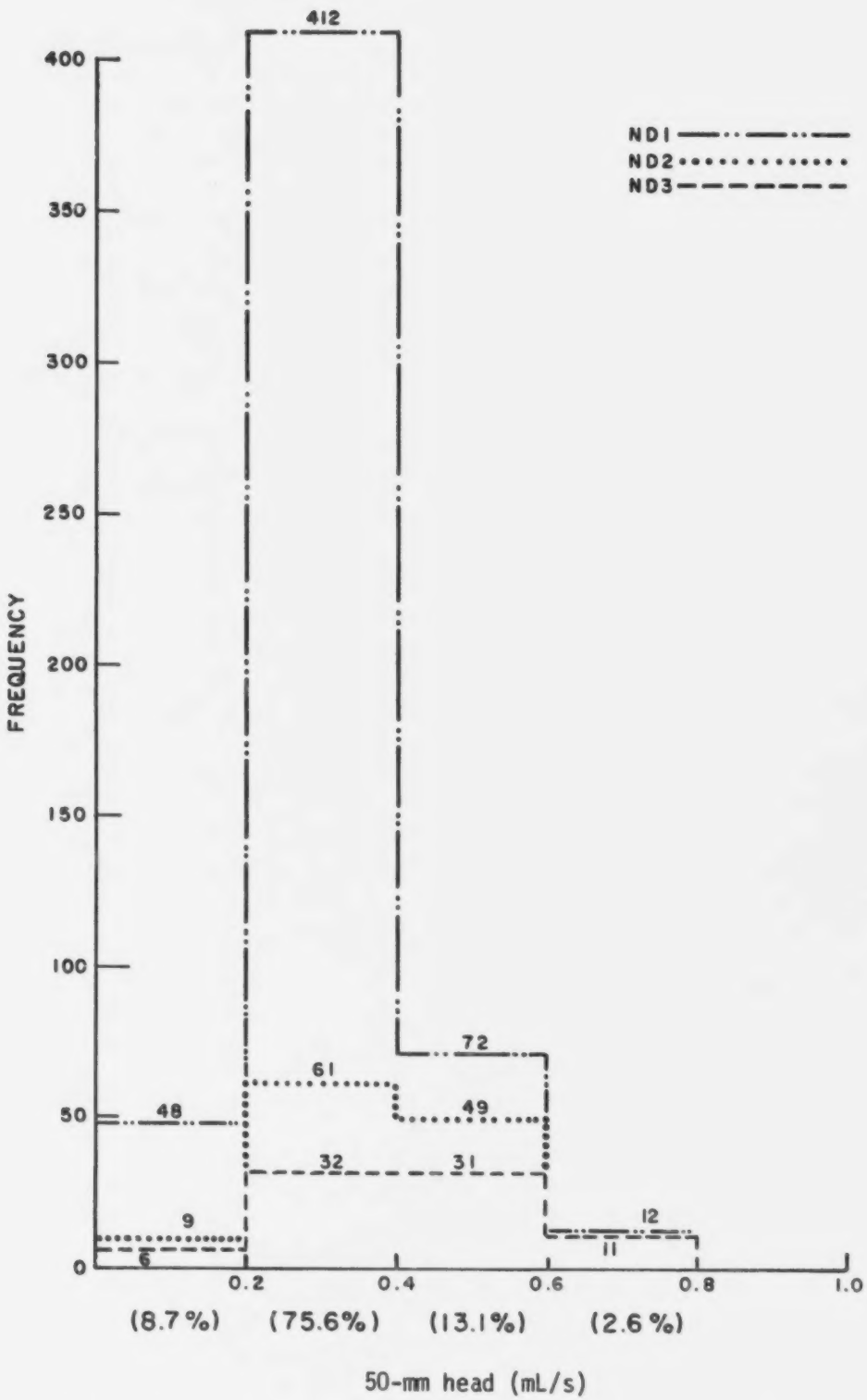


Figure 6-5. - Flow rate data versus frequency of occurrence (sheet 1 of 3).

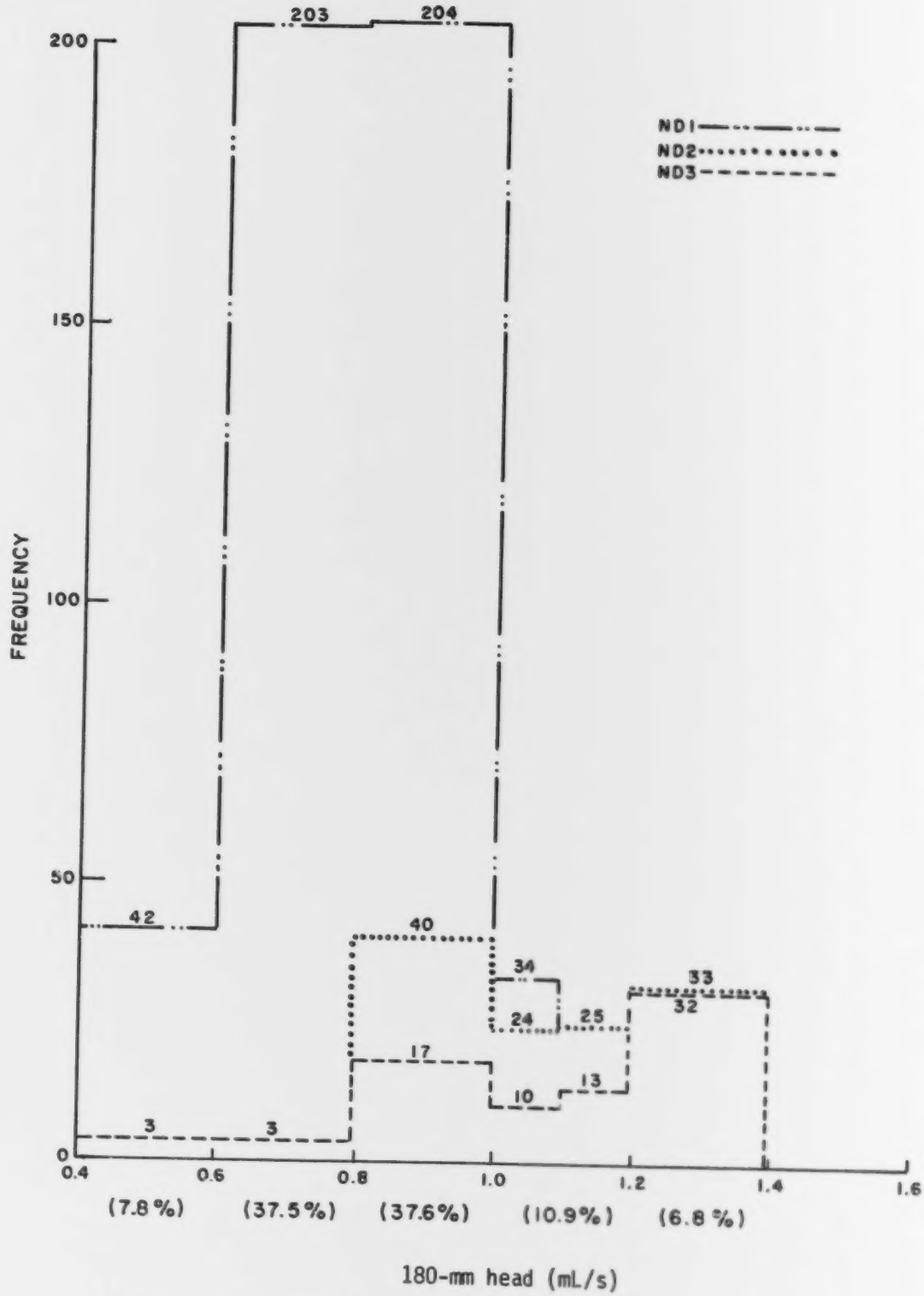
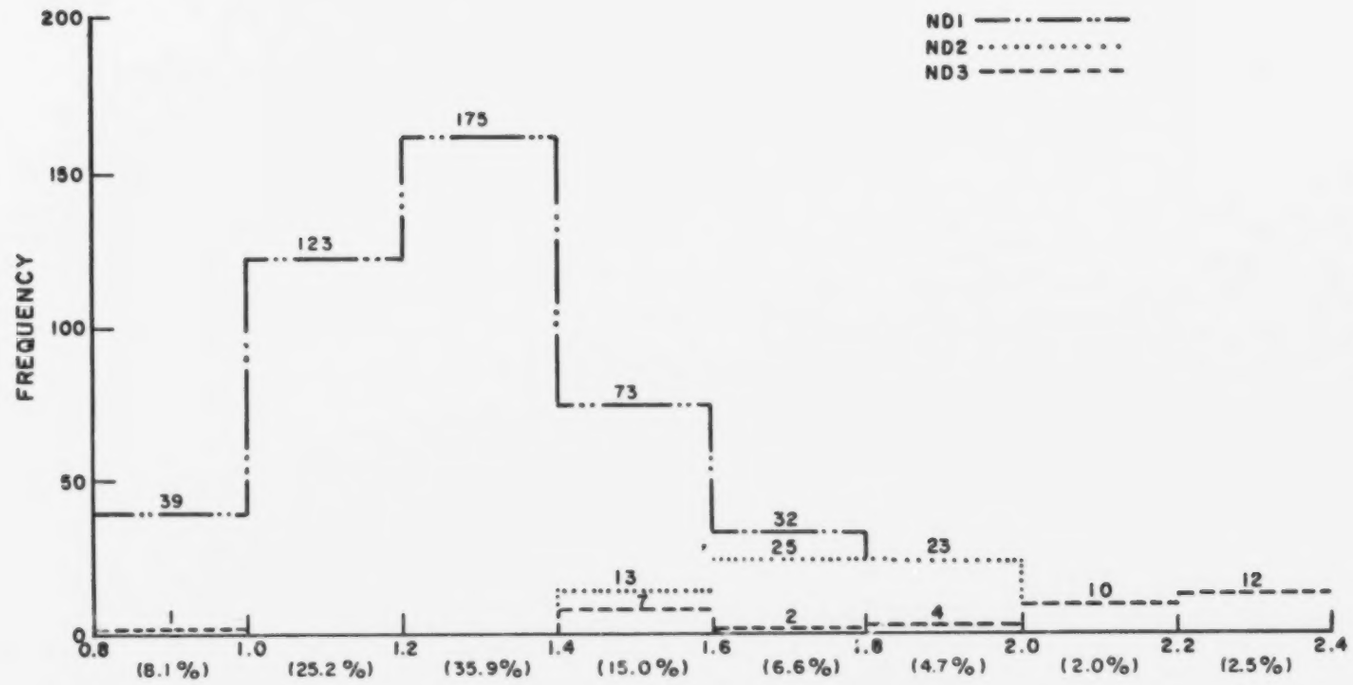


Figure 6-5. - Flow rate data versus frequency of occurrence (sheet 2 of 3).



380-mm head (mL/s)

Figure 6-5. - Flow rate data versus frequency of occurrence (sheet 3 of 3).

ACKNOWLEDGEMENTS

Funds for this study were provided by the McGee Creek Project, Central Arizona Project, and PRESS (Program Related Engineering and Scientific Studies) Project No. DB-30, Site Evaluation of Dispersive Soils. Special appreciation is given to the laboratory chiefs at the projects, Messrs. Bill Pitchford, and Lennis Luginbill (McGee Creek), Rex Burger (Central Arizona), and all the project laboratory technicians and engineers that participated in the test program. This study would not have been possible without their cooperation and valuable suggestions throughout the program.

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APPENDIX A
RED ROCKS PUMPING PLANT

Sample identification		Unified soil class.	Placement d (lb/ft³)	Placement w (%)	Pinhole test data summary Head (mm)	Q_i (mL/s)	Q_f (mL/s)	Turbidity* Initial	Turbidity* Final	Classification	Crumb test	Remarks
AP-1-RRPP	0.0-5.0	SC	115.9	10.8	50	0.30	0.43	C	C	ND1	1	
					180	1.1	1.1	C	C			
					380	1.6	1.7	C	C			
AP-1-RRPP	13.0-49.0	CL	110.7	13.1	50	0.20	0.30	C	C	ND1	1	
					180	0.77	0.77	C	C			
					380	1.3	1.3	C	C			
AP-1-RRPP	56.0-63.0	CL	118.0	10.0	50	0.27	1.1	CL	CL	D2	3	
					180							
					380							
AP-1-RRPP	63.0-74.0	CL	109.3	13.9	50	0.30	0.35	C	C	ND1	1	
					180	0.90	0.92	C	C			
					380	1.6	1.6	C	C			
AP-1-RRPP	84.0-101.0	CL	104.0	16.2	50	0.83	1.1	CL	CL	D1	3	
					180							
					380							
AP-1-RRPP	101.0-106.0	SC-SM			50						1	
					180							
					380							
AP-1-RRPP	106.0-115.0	CL	109.9	12.9	50	0.42	0.78	CL	SC	ND3	2	
					180							
					380							
AP-1-RRPP	115.0-125.0	SC	118.1	9.9	50	0.37	0.33	SC	C	ND3	1	Hole plugged after 3 minutes flow at 180 mm head.
					180	1.23	0.72	SC	SC			
					380							
AP-2-RRPP	0.0-6.0	CL	112.2	12.8	50	0.25	0.47	C	C	ND3	1	
					180	1.2	1.1	SC	SC			
					380	2.0	2.0	CL	CL			
AP-2-RRPP	14.0-41.0	SC-CL	120.4	9.3	50	0.35	0.38	C	C	ND1	1	
					180	0.87	0.92	C	C			
					380	1.3	1.3	C	C			
AP-2-RRPP	41.0-54.0	CL	114.9	11.4	50	0.67	1.2	CL	CL	D1	1	
					180							
					380							
AP-2-RRPP	54.0-74.0	CL	114.9	12.5	50	1.2	1.2	CL	CL	D1	1	
					180							
					380							
AP-2-RRPP	83.0-95.0	CL	111.1	13.4	50	0.32	1.1	SC	SC	D1	1	
					180							
					380							
AP-2-RRPP	95.0-114.0	CL	112.6	12.2	50	0.57	1.1	CL	CL	D1	3	
					180							
					380							
AP-2-RRPP	114.0-117.0	SC	116.8	7.9	50	0.38	0.68	SC	C	ND3	1	
					180	1.6	1.7	C	C			
					380	2.6	2.6	C	C			
AP-2-RRPP	117.0-121.0	SC	122.8	7.9	50	0.23	0.23	C	C	ND1	1	
					180	0.65	0.63	C	C			
					380	1.1	1.1	C	C			
AP-2-RRPP	121.0-125.0	CL	114.7	11.2	50	0.65	0.7	C	C	ND3	1	
					180	1.6	1.5	C	C			
					380	2.6	3.0	C	C			
AP-3-RRPP	0.0-11.0	CL	116.0	14.3	50	0.27	0.27	C	C	ND1	1	
					180	0.75	0.87	C	C			
					380	1.5	1.5	C	C			
AP-3-RRPP	11.0-18.0	SC	125.1	7.1	50	0.30	0.28	C	C	ND1	1	
					180	0.75	0.75	C	C			
					380	1.3	1.2	C	C			
AP-3-RRPP	18.0-28.0	SC-CL	105.2	10.7	50	0.28	0.30	C	C	ND1	1	
					180	0.82	0.87	C	C			
					380	1.4	1.4	C	C			
AP-3-RRPP	28.0-35.0	CL	119.2	12.8	50	0.32	1.1	CL	CL	D2	2	
					180							
					380							
AP-4-RRPP	0.0-9.0	CL-SC	117.9	10.1	50	0.33	0.32	C	C	ND1	1	
					180	0.90	0.87	C	C			
					380	1.4	1.4	C	C			
AP-4-RRPP	14.0-19.0	SC	122.1	8.2	50	0.42	1.1	SC	SC	D1	1	
					180							
					380							
AP-4-RRPP	19.0-28.0	CL-SC	115.4	10.6	50	0.27	1.0	CL	CL	D2	2	
					180							
					380							
AP-4-RRPP	28.0-35.0	SC-CL	115.2	11.0	50	0.25	0.40	SC	C	ND1	1	
					180	0.88	0.90	SC	C			
					380	1.37	1.37	SC	C			

* D = Dark/distinctly cloudy CL = Cloudy SC = slightly cloudy C = Clear

APPENDIX B
BRADY PUMPING PLANT

Sample Identification		Unified soil class.	Pinhole test data summary: Placement d (lb/ft³)	w (%)	Head (cm)	Q_i (mL/s)	Q_f (mL/s)	Turbidity* initial	Turbidity* final	Classification	Crumb test	Remarks
AP-1-BPP	0.0-6.0	SC-CL	125.5	9.8	50	0.27	0.28	C	C	ND1	1	
					180	0.63	0.63	C	C			
					380	1.0	1.0	C	C			
AP-1-BPPP	6.0-11.0	SC	118.2	12.1	50	0.23	0.25	C	C	ND1	1	
					180	0.60	0.58	C	C			
					380	0.95	0.93	C	C			
AP-1-BPP	11.0-32.0	SC	122.5	10.7	50	0.20	0.98	SC	SC	ND4	3	
					180	2.53	2.52	SC	SC			
					380							
AP-1-BPP	32.0-37.0	SC	114.0	13.4	50	0.38	0.58	SC	SC	ND4	4	
					180	2.2	2.4	SC	SC			
					380							
AP-1-BPP	37.0-58.0	SC	119.8	11.7	50	0.55	1.1	SC	SC	D2	3	
					180							
					380							
AP-1-BPP	63.0-91.0	SC	120.2	11.7	50	0.58	1.0	SC	SC	D1	3	
					180							
					380							
AP-1-BPP	91.0-98.0	SC-CL	119.0	12.1	50	0.58	1.1	SC	SC	D2	3	
					180							
					380							
AP-1-BPP	98.0-125.0	CL	116.7	13.2	50	0.90	1.2	SC	SC	D1	4	
					180							
					380							
AP-2-BPP	7.0-36.0	SC	119.4	12.3	50	0.23	0.23	C	C	ND1	1	
					180	0.57	0.58	C	C			
					380	0.93	0.93	C	C			
AP-2-BPP	36.0-58.0	SC	108.9	11.3	50	0.17	1.2	CL	SC	D2	4	
					180							
					380							
AP-2-BPP	66.0-90.0	CL-SC	109.9	13.7	50	0.53	1.1	CL	CL	D1	3	
					180							
					380							
AP-2-BPP	90.0-106.0	CL-SC	121.0	12.6	50	0.32	0.45	SC	CL	ND3	3	
					180	1.07	1.03	CL	CL			
					380							
AP-2-BPP	106.0-119.0	SC	120.1	9.9	50	0.23	1.0	CL	CL	D2	3	
					180							
					380							
AP-2-BPP	119.0-125.0	CL	110.4 (107.5)	17.1 (15.2)	50	0.52	0.87	CL	SC	ND4	3	
					180	2.0	2.0	SC	SC			
					380							
AP-3-BPP	0.0-35.0	SC-SM	127.8 (122.6)	9.6 (8.1)	50	0.37	0.37	C	C	ND1	2	
					180	0.83	0.83	C	C			
					380	1.4	1.3	C	C			

* D = Dark/distinctly cloudy CL = Cloudy SC = Slightly cloudy C = Clear

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APPENDIX C
TUCSON AQUEDUCT - REACH 2

Sample Identification		Unified soil class.	Placement d (lb/ft³)	w (%)	Pinhole test data summary Head (mm)	Q_1 (mL/s)	Q_2 (mL/s)	Turbidity* Initial	Final	Classification	Crumb test	Remarks
AP-101-TA2	6.0-8.5	SC	120.5	8.5	50	0.27	0.27	C	C	ND1	2	- No. 30 soil
					180	0.70	0.72	C	C			
					380	1.1	1.2	C	C			
AP-106-TA2	24.0-26.0	SC	117.6	9.6	50	0.33	0.32	C	C	ND1	1	- No. 30 soil
					180	0.85	0.83	C	C			
					380	1.4	1.4	C	C			
AP-108-TA2	10.0-21.0	SC	128.1	8.6	50	0.38	0.38	C	C	ND1	1	
					180	0.88	0.88	C	C			
					380	1.6	1.6	C	C			
AP-109-TA2	10.0-23.0	SC	128.1	9.4	50	0.42	0.42	C	C	ND1	1	
					180	0.88	0.88	C	C			
					380	1.5	1.5	C	C			
AP-111-TA2	16.0-18.0	SC-SC	117.1	15.9	50	0.22	0.2	C	C	ND1	1	- No. 30 soil
					180	0.62	0.65	SC	SC			
					380	1.1	1.2	SC	SC			
AP-112-TA2	0.0-4.0	CL-SC	117.9	10.9	50	0.27	0.27	SC	SC	ND1	2	
					180	0.67	0.72	C	C			
					380	1.2	1.1	C	C			
	17.0-20.0	SC	125.8	8.9	50	0.35	0.40	SC	C	ND1	2	- No. 30 soil
					180	0.90	0.95	C	C			
					380	1.4	1.5	C	C			
AP-113-TA2	0.0-23.0	CL	104.8	18.7	50	0.28	0.37	C	C	ND1	1	
					180	0.83	0.85	C	C			
					380	1.3	1.3	C	C			
AP-115-TA2	0.0-6.0	CL	111.4	15.3	50	0.22	0.22	C	C	ND1	2	
					180	0.62	0.60	C	C			
					380	1.0	1.0	C	C			
	6.0-13.0	CL	105.2	16.2	50	0.28	0.27	C	C	ND1	1	
					180	0.72	0.72	C	C			
					360	1.2	1.2	C	C			
	13.0-28.0	CL-SC	107.6	11.9	50	0.27	0.27	C	C	ND1	1	
					180	0.68	0.67	C	C			
					380	1.0	1.0	C	C			
AP-116-TA2	0.0-10.0	CL	108.9	11.5	50	0.30	0.30	C	C	ND1	1	
					180	0.80	0.80	C	C			
					380	1.3	1.3	C	C			
	18.0-23.0	CL-SC	116.7	12.5	50	0.27	0.27	C	C	ND1	1	
					180	0.68	0.67	C	C			
					380	1.1	1.1	C	C			
AP-118-TA2	0.0-7.0	CL	112.5	12.8	50	0.33	0.35	C	C	ND2	1	
					180	0.98	1.1	C	C			
					380	1.7	1.8	C	C			
	16.0-23.0	SC	119.3	9.3	50	0.25	0.25	C	C	ND1	1	
					180	0.63	0.63	C	C			
					380	1.1	1.1	C	C			
AP-120-TA2	8.0-30.0	CL	110.2	13.4	50	0.22	1.1	CL	CL	02	2	
					180							
					360							
	8.0-30.0	CL	110.2	13.4	50	0.22	0.97	CC	CL	02		Replicate test (see above)
					180							
					380							
AP-122-TA2	0.0-7.0	CL	107.2	13.5	50	0.27	0.27	C	C	ND1	1	
					180	0.68	0.67	C	C			
					380	1.1	1.1	C	C			
	7.0-13.0	CL	116.9	10.7	50	0.25	0.33	C	C	ND1	1	
					180	0.80	0.80	C	C			
					380	1.3	1.3	C	C			
	13.0-35.0	CL	102.1	15.1	50	0.23	0.52	CL	SC	ND4	1	
					180	2.0	2.6	CL	CL			
					380							
	13.0-35.0	CL	102.1	15.1	50	0.32	0.73	SC	CL	ND4		- No. 30 soil used for retest (see above)
					180	1.9	2.5	CL	CL			
					380							
AP-123-TA2	0.0-5.0	CL	106.0	15.2	50	0.22	0.33	C	C	ND1	1	Repunched hole at 2 min. flow at 50 mm head
					180	0.83	0.83	C	C			
					380	1.4	1.4	C	SC			
	5.0-11.0	SC-CL	110.5	12.8	50	0.33	0.32	C	C	ND1	1	
					180	0.78	0.78	C	C			
					380	1.3	1.3	C	C			
	11.0-30.0	CL	105.5	15.0	50	0.13	0.38	C	C	ND1	1	
					180	0.98	0.98	C	C			
					380	1.6	1.6	C	C			
AP-124-TA2	0.0-5.0	CL	102.5	15.6	50	0.43	0.38	C	C	ND1	1	
					180	0.97	0.97	C	C			
					380	1.6	1.6	C	C			

APPENDIX C - Continued
TUCSON AQUEDUCT - REACH 2

Sample identification		Unified soil class.	Placement γ_d (lb/ft^3)	w (%)	Pinhole test data summary: Head (mm)	Q_i (mL/s)	Q_f (mL/s)	Turbidity[a] Initial	Turbidity[a] Final	Classification	Crumb test	Remarks
	5.0-11.0	SC	116.4	10.9	50	0.27	0.27	C	C	ND1	1	
					180	0.67	0.67	C	C			
					380	1.2	1.2	C	C			
AP-124-TA2	11.0-13.0	SC	119.0	9.5	50	0.23	0.23	C	C	ND1	1	
					180	0.65	0.65	C	C			
					380	1.1	1.0	C	C			
	13.0-30.0	SC-CL	107.9	13.9	50	0.32	0.32	C	C	ND2	1	
					180	0.90	0.90	C	C			
					380	1.6	1.6	C	C			
AP-126-TA2	0.0-10.0	CL	106.0	14.9	50	0.32	0.33	C	C	ND2	1	
					180	1.2	1.3	C	C			
					380	2.2	2.2	C	C			
	6.0-10.0	SC	114.3	11.3	50	0.33	0.33	C	C	ND1	1	
					180	0.85	0.85	C	C			
					380	1.4	1.4	C	C			
	20.0-30.0	CL			50	0.37	0.43	SC	SC	ND4	1	
					180	2.1	2.1	CL	CL			
					380							
AP-127-TA2	0.0-6.0	CL	114.3	11.6	50	0.30	0.30	C	C	ND1	1	
					180	0.77	0.77	C	C			
					380	1.2	1.2	C	C			
	6.0-13.0	SC		9.3	50	0.30	0.30	C	C	ND1	1	
					180	0.77	0.77	C	C			
					380	1.3	1.3	C	C			
	13.0-18.0	CL-SC	109.7	13.5	50	0.47	1.1	CL	CL	D2	1	
					180							
					380							
	18.0-25.0	SC	115.1	10.6	50	0.40	0.40	C	C	ND1	1	
					180	1.0	1.0	C	C			
					380	1.6	1.6	C	C			
	25.0-30.0	CL	113.8	11.7	50	0.30	0.98	CL	CL	ND4	1	
					180			CL	CL			
					380							
AP-129-TA2	0.0-6.0	CL	107.2	14.8	50	0.25	0.92	SC	CL	ND4	1	
					180	2.6	2.3	CL	CL			
					380							
	6.0-9.0	SC	107.4	8.4	50	0.33	0.33	C	C	ND1	1	
					180	0.82	0.82	C	C			
					380	1.3	1.3	C	C			
	9.0-22.0	CL-SC	113.9	14.8	50	0.33	0.33	SC	C	ND1	1	
					180	0.80	0.83	SC	C			
					380	1.3	1.3	C	C			
AP-129-TA2	22.0-30.0	SC-CL	118.1	10.5	50	0.42	0.47	CL	CL	ND3	1	
					180	1.0	0.98	CL	CL			
					380	1.6	3.5	CL	CL			
AP-130-TA2	0.0-0.6	CL	112.1	14.3	50	0.13	0.33	SC	CL	ND3	1	
					180	0.87	0.97	CL	CL			
					380	2.2	2.8	CL	CL			
	6.0-13.0	CL	112.0	12.1	50	0.33	0.28	C	C	ND1	1	
					180	0.78	0.78	C	C			
					380	1.3	1.3	C	C			
	13.0-17.0	CL	NA	NA	50	0.30	0.52	C	SC	ND3	1	
					180	1.3	1.3	SC	SC			
					380	2.5	2.7	SC	SC			
	27.0-35.0	CL	110.4	13.1	50	0.35	1.1	CL	CL	D1	3	
					180							
					380							
AP-131-TA2	0.0-6.0	CL	107.2	15.1	50	0.33	0.80	CL	C	ND4	1	
					180	1.8	2.1	C	C			
					380							
	6.0-8.0	CL-SC	114.9	11.5	50	0.25	0.35	C	C	ND1	1	
					180	0.88	0.88	C	C			
					380	1.4	1.4	C	C			
	8.0-10.5	SC	124.9	7.4	50	0.28	0.23	C	C	ND1	1	
					180	0.67	0.67	C	C			
					380	1.2	1.1	C	C			
	10.5-20.0	CL-SC	118.0	13.1	50	0.32	0.30	SC	C	ND1	1	
					180	0.77	0.80	C	C			
					380	1.3	1.4	C	C			
	20.0-26.0	SC	117.0	10.5	50	0.33	1.1	CL	CL	D2	1	- No. 30 soil
					180							
					380							
	26.0-35.0	CL	117.4	12.7	50	0.28	1.1	CL	CL	D2	2	
					180							
					380							

APPENDIX C - Continued
TUCSON AQUEDUCT - REACH 2

Sample identification		Unified soil class.	Pinhole test data summary: Placement d (lb/ft³)	w (%)	Head (mm)	Q_i (mL/s)	Q_e (mL/s)	Turbidity* initial	final	Classification	Crumb test	Remarks
AP-133-TA2	16.0-25.0		117.0	12.5	50	0.35	1.1	CL	CL	D2	1	
					180							
					380							
	30.0-45.0		120.1	11.0	50	0.55	1.1	CL	CL	D2	2	
					180							
					380							
AP-137-TA2	0.0-5.0	CL	124.2	10.0	50	0.37	0.55	C	C	ND2	1	
					180	1.2	1.2	C	C			
					380	1.9	2.0	C	C			
	5.0-9.0	CL	117.4	12.8	50	0.63	0.42	CL	SC	ND3	1	
					180	1.0	1.0	SC	SC			
					380	1.6	1.6	SC	SC			
	9.0-19.0	SC	118.7	7.7	50	0.33	0.40	CL	CL	ND3	1	- No. 30 soil
					180	1.25	1.23	CL	CL			
					380							
	19.0-25.0	CL	119.5	11.7	50	0.38	1.1	CL	CL	D1	2	
					180							
					380							
	28.0-35.0	CL	116.0	12.2	50	0.20	0.32	C	C	ND1	1	
					180	0.83	0.82	C	C			
					380	1.33	1.33	C	C			
AP-139-TA2	18.0-35.0	CL	123.5	7.9	50	0.32	0.30	C	C	ND1	1	
					180	0.80	0.78	C	C			
					380	1.3	1.3	C	C			

* D = Dark/distinctly cloudy CL = Cloudy SC = Slightly cloudy C = Clear

APPENDIX D
TUCSON AQUEDUCT - REACH 3

Sample identification		Unified soil class.	Placement d (lb/ft³)	Placement w (%)	Pinhole test data summary Head (mm)	Q_1 (mL/s)	Q_2 (mL/s)	Turbidity[a] Initial	Turbidity[a] Final	Classification	Crumb test	Remarks
AP-101-TA3	13.0-20.0		121.2	11.3	50	0.30	0.33	C	C	ND1	1	
					180	0.88	0.88	C	C			
					380	1.4	1.4	C	C			
AP-103-TA3	10.0-14.0		119.8	9.0	50	0.25	0.30	C	C	ND1	1	Holes repunched to start flow at 50 mm head
					180	0.82	0.82	C	C			
					380	1.4	1.4	C	C			
	14.0-17.0		124.5	7.7	50	0.28	0.28	C	C	ND1	1	
					180	0.80	0.80	C	C			
					380	1.4	1.4	C	C			
	17.0-25.0		121.4	10.5	50	0.27	0.30	SC	C	ND3	1	
					180	0.80	0.92	SC	SC			
					380	1.5	1.6	SC	SC			
AP-105-TA3	9.0-25.0		121.5	10.9	50	0.28	0.28	C	C	ND1	1	
					180	0.73	0.73	C	C			
					380	1.2	1.2	C	C			
AP-107-TA3	7.0-13.0		117.9	11.1	50	0.28	0.28	C	C	ND1	1	
					180	0.82	0.82	C	C			
					380	1.4	1.4	C	C			
	13.0-25.0		122.8	10.6	50	0.33	0.35	C	C	ND1	1	
					180	0.88	0.88	C	C			
					380	1.4	1.4	C	C			
AP-109-TA3	8.0-21.0		120.0	10.7	50	0.37	0.35	C	C	ND1	1	
					180	0.92	0.92	C	C			
					380	1.4	1.4	C	C			
	21.0-25.0		116.4	12.0	50	0.43	1.0	CL	CL	D1	2	
					180							
					380							
	25.0-30.0		125.8	9.4	50	0.42	0.58	CL	CL	ND3	3	
					180	0.93	1.2	CL	CL			
					380							
AP-110-TA3	0.0-6.0		117.3	8.2	50	0.33	0.32	C	C	ND1	1	
					180	0.77	0.75	C	C			
					380	1.2	1.2	C	C			
	6.0-19.0		123.5	9.8	50	0.30	0.33	SC	C	ND1	1	
					180	0.87	0.85	SC	C			
					380	1.3	1.3	C	C			
	19.0-25.0		121.2	10.7	50	0.40	1.1	CL	CL	D1	2	
					180							
					380							
AP-111-TA3	10.0-18.0		114.7	13.0	50	0.23	0.35	C	C	ND2	1	
					180	0.95	0.95	C	C			
					380	1.6	1.7	C	C			
	18.0-25.0		125.2	17.8	50	0.33	0.97	SC	SC	ND4	2	
					180	2.2	2.5	SC	SC			
					380							
	25.0-30.0		115.4	12.7	50	0.25	0.35	C	C	ND2	1	
					180	0.90	0.88	C	C			
					380	1.6	1.6	SC	C			
AP-113-TA3	6.0-18.0		116.3	10.3	50	0.25	0.37	C	C	ND1	1	
					180	0.93	0.93	C	C			
					380	1.5	1.5	C	C			
	18.0-25.0		117.7	10.8	50	0.6	1.1	CL	CL	D2	3	
					180							
					380							
AP-114-TA3	10.0-18.0		121.0	13.4	50	0.25	0.30	C	C	ND1	1	
				8.9	180	0.73	0.73	C	C			
					380	1.2	1.2	C	C			
	18.0-23.0		123.4	10.3	50	0.25	0.57	C	C	ND3	1	
					180	1.4	1.4	SC	C			
					380							
AP-116-TA3	9.0-19.0		126.7	8.1	50	0.32	0.32	C	C	ND1	1	
					180	0.80	0.77	C	C			
					380	1.4	1.3	C	C			
AP-118-TA3	12.0-19.0		124.3	7.7	50	0.25	0.35	C	C	ND1	1	
					180	0.90	0.88	C	C			
					380	1.4	1.4	C	C			
	12.0-19.0		123.0	7.7	50	0.20	0.30	C	C	ND1		Replicate test - apparent shrinkage potential/expansion soil
					180	0.45	0.45	C	C			
					380	0.77	0.72	C	C			
AP-136-TA3	0.0-19.0		115.9	12.1	50	0.83	1.1	CL	CL	D1	1	
					180							
					380							
	19.0-25.0		111.6	15.1	50	0.37	0.38	C	C	ND2	1	
					180		1.0	C	C			
					380		1.7	C	C			

APPENDIX D
TUCSON AQUEDUCT - REACH 3

Sample identification		Unified soil class.	Placement γ_d (lb/ft^3)	Placement w (%)	Pinhole test data summary: Head (cm)	Q_1 (mL/s)	Q_2 (mL/s)	Turbidity* initial	Turbidity* final	Classification	Crumb test	Remarks
AP-137-TA3	0.0-12.0		101.5	21.0	50	0.20	0.58	SC	SC	ND3	1	
					100	1.4	1.5	SC	C			
					380							
AP-137-TA3	12.0-16.0		108.0	20.2	50	0.75	1.2	CL	CL	D1	1	
				18.2	100							
					380							
AP-138-TA3	6.0-18.0		116.5	10.7	50	0.27	0.32	C	C	ND1	1	
					100	0.80	0.80	C	C			
					380	1.3	1.3	C	C			
	18.0-30.0		110.5	14.6	50	0.73	1.2	CL	CL	D1	3	
					100							
					380							
AP-139-TA3	10.0-20.0		118.0	9.6	50	0.28	0.28	C	C	ND1	1	
					100	0.78	0.78	C	C			
					380	1.3	1.3	C	C			
AP-141-TA3	28.0-30.0		113.4		50	0.37	1.1	SC	SC	D1	3	
					100							
					380							

* D = Dark/distinctly cloudy CL = Cloudy SC = Slightly cloudy C = Clear

APPENDIX E
CLIFF DAM - DOWNSTREAM AREA

Sample identification		Unified soil class.	Placement d (lb/ft³)	w (%)	Pinhole test data summary Head (mm)	Q_i (mL/s)	Q_f (mL/s)	Turbidity* Initial	Final	Classification	Crumb test	Remarks
TP-110-CSD	1.0-6.0	GC	111.4	11.4	50	0.25	0.53	SC	C	ND3	1	Hole repunched at 180 mm ND after clogging occurred
					180	1.2	0.22	C	C			
					380	1.6	0.4	SC	SC			
TP-118-CSD	1.0-4.0	CL	109.9	18.8	50	0.40	0.28	SC	C	ND3	1	
					180	1.1	1.2	SC	SC			
					380							
	1.0-4.0	CL	105.3	18.8	50	0.47	0.52	SC	SC	ND3		Replicate test
					180	1.1	0.27	SC	CL			
					380							
TP-123-CSD	1.5-4.0	CH	107.4	17.9	50	0.42	0.37	C	C	ND3	1	Hole repunched three times to initiate flow at 50 mm head
					180	1.0	1.1	SC	C			
					380							
TP-210-CSD	0.0-3.0	CL	120.2	10.6	50	0.30	1.0	CL	SC	D2	3	
					180							
					380							
TP-218-CSD	0.0-3.0	CH-SC	106.5	17.6	50	0.22	0.32	C	C	ND1	1	Hole repunched three times during first 5 minutes of flow at 50 mm head
					180							
					380							
TP-302-CSD	0.0-1.5	CL	120.1	11.5	50	0.83	0.87	C	C	ND3	1	* * * could be ND2 based on clear flow at 380 ND and 1 mm final pinhole
					180	1.2	1.5	C	C			
					380	2.1	2.2	C	C			
TP-308-CSD	0.0-5.0	CL-SC	108.3	15.1	50	0.48	1.1	CL	CL	D2	2	
					180							
					380							
TP-408-CSD	0.0-4.0	CL	117.3	13.6	50	0.82	1.1	CL	CL	D1	3	
					180							
					380							
TP-413-CSD	6.5-10.0	SC	108.0	16.2	50	0.32	0.37	C	C	ND1	1	
					180	0.87	0.88	C	C			
					380	1.4	1.5	C	C			
TP-519-CSD	11.5-18.0	CL-SC	114.0	11.8	50	0.60	1.1	CL	CL	D1	1	
					180							
					380							
TP-607-CSD	4.0-17.0	CL	99.8	22.5	50	0.35	0.47	C	SC	ND3	1	
					180	1.2	1.2	SC	C			
					380							
TP-608-CSD	1.5-8.0	CL-SC	106.9	15.8	50	0.23	0.38	SC	SC	ND3	1	- No. 30 soil - soil erosion sketch shows trumpeting disc/original pinhole V/S
					180	0.95	1.1	CL	CL			
					380							
AH-617-CSD	0.0-4.5	CL	114.1	15.1	50	0.20	0.37	C	C	ND3	1	* * * could be ND1 based on 1 mm final diameter hole clear flow
					180	1.0	1.0	C	C			
					380							
AH-619-CSD	3.0-5.0	CL	107.4	16.2	50	0.18	0.32	C	C	ND1	1	
					180	0.95	0.95	C	C			
					380	1.5	1.6	C	C			

* D = Dark/distinctly cloudy CL = Cloudy SC = Slightly cloudy C = Clear

APPENDIX F
TUCSON AQUEDUCT - REACH 1

Sample identification	Unified soil class.[a]	Pinhole test data summary: Placement d (lb/ft³)	Placement w (%)	Head (mm)	Q_i (mL/s)	Q_f (mL/s)	Turbidity[a] Initial	Turbidity[a] Final	Classification	Crumb test	Remarks
AP-137-TA1 Sta 349+20 5.0-10.0	CL	114.2	11.5	50 180 380	0.28 2.2	0.90 2.8	CL CL	SC C	ND4	1	
10.0-15.0	SC-SM	119.7	11.2	50 180 380	0.47	1.1	CL	SC	D1	3	
15.0-20.0	SC			50 180 380	0.58	1.2	CL	CL	D1	4	- No. 30 soil
20.0-25.0	SC	118.0	10.8	50 180 380	0.23	1.1	SC	SC	D1	3	- No. 30 soil
25.0-30.0	CL	113.6	11.3	50 180 380	0.30	1.1	SC	SC	D1	3	- No. 30 soil
30.0-35.0	SC	115.4	10.0	50 180 380	0.43	1.1	CL	CL	D1	3	- No. 30 soil
AP-138-TA1 Sta 359+70 0.0-5.0	SC-CL	119.1	9.5	50 180 380	0.43	1.1	CL	CL	D1	3	- No. 30 soil
5.0-10.0	SC-SC	115.9	10.6	50 180 380	0.32	1.1	SC	SC	D1	2	- No. 30 soil
10.0-15.0	SC	122.2	8.7	50 180 380	0.37	1.1	SC	SC	D1	3	- No. 30 soil
15.0-20.0	SC	120.8	10.1	50 180 380	0.20	1.1	SC	SC	D1	3	- No. 30 soil
20.0-25.0	SC	118.5	10.0	50 180 380	0.38	1.1	SC	SC	D1	3	- No. 30 soil
25.0-30.0	SC	119.6	10.0	50 180 380	0.35	1.1	SC	SC	D1	3	
30.0-35.0	SC	114.0	11.3	50 180 380	0.30	1.1	SC	SC	D1	3	- No. 30 soil
AP-138-TA1 Sta 359+70 35.0-40.0	SC	121.2	9.1	50 180 380	0.40	1.1	SC	SC	D1	3	- No. 30 soil
AP-139-TA1 Sta 366+50 0.0-5.0	SC	124.9	8.6	50 180 380	0.43	1.1	CL	CL	D1	3	- No. 30 soil
0.5-10.0	SC-CL	120.7	10.7	50 180 380	0.30 0.77 1.3	0.30 0.77 1.3	C C C	C C C	ND1	1	
10.0-15.0	SC	124.8	8.7	50 180 380	0.28 0.75 1.2	0.28 0.75 1.2	C C C	C C C	ND1	1	- No. 30 soil
15.0-20.0	SC	118.8	10.9	50 180 380	0.32	1.1	SC	SC	D2	3	- No. 30 soil
20.0-25.0	SC	116.7	10.0	50 180 380	0.30	1.1	SC	SC	D2	3	- No. 30 soil
25.0-30.0	SC	115.2	11.8	50 180 380	0.30 0.72 1.1	0.30 0.72 1.1	SC SC SC	SC SC SC	ND1	1	- No. 30 soil
30.0-35.0	SC	118.0	11.0	50 180 380	0.32	1.1	SC	SC	D1	3	- No. 30 soil
35.0-40.0	SC	119.3	11.5	50 180 380	0.30	1.1	CL	SC	D1	4	- No. 30 soil
40.0-45.0	SC	118.8	11.3	50 180 380	0.30	1.1	CL	SC	D1	3	- No. 30 soil
45.0-50.0	SC	113.3	13.1	50 180 380	0.42	1.1	CL	CL	D1	3	- No. 30 soil
AP-140-TA1 5.0-10.0 Sta 367+30, 40 LT	SC	119.6	9.4	50 180 380	0.30	1.1	SC	SC	D2	3	- No. 30 soil

APPENDIX F - Continued
TUCSON AQUEDUCT - REACH 1

Sample identification		Unified soil class.	Pinhole test data summary: Placement d (lb/ft³)	w (%)	Head (mm)	Q_i (mL/s)	Q_f (mL/s)	Turbidity* Initial	Turbidity* Final	Classification	Crumb test	Remarks
	10-15.0	SC	123.0	10.0	50 180 380	0.30	1.1	SC	SC	D2	3	- No. 30 soil
AP-140-TA1 Sta 375+30, 40 LT	15.0-25.0	SC	120.3	9.3	50 180 380	0.30	1.1	SC	SC	D2	3	- No. 30 soil
	20.0-25.0	SC	118.6	11.1	50 180 380	0.35	1.1	SC	SC	D1	3	- No. 30 soil
	25.0-30.0	SC	117.5	10.8	50 180 380	0.30	1.1	SC	SC	D1	3	- No. 30 soil
	30.0-35.0	SC	121.0	9.9	50 180 380	0.40	1.1	SC	SC	D1	3	- No. 30 soil
	35.0-40.0	SC	119.7	10.1	50 180 380	0.37	1.1	SC	SC	D1	3	- No. 30 soil
AP-141-TA1 Sta 375+30 50 RT	0.0-5.0	SC	126.1	8.4	50 180 380	0.48	1.1	CL	CL	D1	2	- No. 30 soil
	5.0-10.0	SC-CL	117.4	11.6	50 180 380	0.30 0.82 1.3	0.30 0.82 1.3	C C C	C C C	MD1	1	- No. 30 soil
	10.0-15.0	SC	116.7	9.9	50 180 380	0.30	1.1	SC	SC	D2	3	- No. 30 soil
	15.0-20.0	SC	119.5	10.9	50 180 380	0.36	1.1	SC	SC	D1	3	- No. 30 soil
	20.0-25.0	SC	116.7	11.2	50 180 380	0.30	1.1	C	SC	D2	3	- No. 30 soil
	25.0-30.0	SC	114.7	12.5	50 180 380	0.43	1.1	SC	SC	D1	2	- No. 30 soil
	30.0-35.0	SC	119.3	9.8	50 180 380	0.45	1.1	SC	SC	D1	3	- No. 30 soil
	35.0-40.0	SC	118.0	11.0	50 180 380	0.32	1.1	SC	SC	D2	3	- No. 30 soil
AP-142-TA1 Sta 382+15	0.0-5.0	SC-SM	122.8	8.1	50 180 380	0.53	1.1	SC	SC	D1	3	- No. 30 soil
	5.0-10.0	SC	119.1	9.8	50 180 380	0.30 0.82 1.4	0.30 0.82 1.4	C C C	C C C	MD1	1	- No. 30 soil
	10.0-15.0	SC	122.6	9.2	50 180 380	0.30 0.83 1.4	0.30 0.83 1.4	C C C	C C C	MD1	1	- No. 30 soil
	15.0-20.0	SC	118.8	10.5	50 180 380	0.30 0.93 2.3	0.35 1.0 3.8	SC C SC	C SC SC	MD3	1	- No. 30 soil
	20.0-25.0	SC	117.1	11.1	50 180 380	0.37	1.1	SC	SC	D1	2	- No. 30 soil
	25.0-30.0	SC	111.2	13.9	50 180 380	0.30 1.8	0.57 2.6	SC SC	SC SC	MD4	1	- No. 30 soil
	30.0-35.0	SC	119.3	8.9	50 180 380	0.35	1.1	SC	SC	D1	4	- No. 30 soil
	35.0-40.0	SC	114.7	11.9	50 180 380	0.55	1.1	SC	SC	D1	3	- No. 30 soil

* D = Dark/distinctly cloudy CL = Cloudy SC = Slightly cloudy C = Clear

APPENDIX G
TUCSON AQUEDUCT - REACH 1 - BORROW AREA

Sample identification		Unified soil class.	Placement d (lb/ft³)	Placement w (%)	Pinhole test data summary: Head (mm)	Q_i (mL/s)	Q_f (mL/s)	Turbidity[a] Initial	Turbidity[a] Final	Classification	Crumb test	Remarks
AP-134-TA1	5.0-17.0	SC	125.3	9.1	50 180 380	0.33 1.8	0.65 0.80	C SC	SC SC	ND3	2	Apparent plugging at 180 mm head
	17.0-33.0	SC	117.4	9.6	50 180 380	0.40 1.4 2.1	0.62 1.3 1.7	SC C C	C C C	ND3	1	
	33.0-50.0	SC	117.8	11.1	50 180 380	0.32 2.4	0.88 2.5	SC SC	SC SC	ND4	1	
	50.0-65.0	SC	113.4	11.6	0 180 380	0.38 0.93 1.4	0.40 0.90 1.4	SC C C	C C C	ND1	1	
AP-135-TA1	0.0-8.0	SC-CL	126.4	9.2	50 180 380	0.30	1.1	SC	SC	D2	2	
	8.0-18.0	SC-CL	122.6	10.4	50 180 380	0.35 0.88 2.7	0.33 1.2	C C	C SC	ND3	1	
	18.0-22.0	SC-SM	119.2	9.9	50 180 380	0.52	1.1	SC	SC	D2	2	
	22.0-30.0	SC-CL	116.3	10.5	50 180 380	0.27	1.1	SC	C	D2	4	
	30.0-36.0	SC-SM	121.4	10.5	50 180 380	0.38 1.8	1.1 2.3	SC CL	SC CL	D2	3	
	36.0-40.0	SC	120.9	9.6	50 180 380	0.55	1.1	CL	CL	D1	3	
	0.0-9.0	SC-CL	123.7	8.9	50 180 380	0.42	1.1	CL	SC	D1	3	
	13.0-19.0	SC-CL	117.6	11.4	50 180 380	0.35	1.1	SC	SC	D2	3	
	19.0-34.0	SC-CL	115.9	10.5	50 180 380	0.32 1.0	0.65 2.5	SC SC	SC SC	ND4	3	
AP-201-TA1	21.0-28.0	CL	117.9	11.1	50 180 380	0.25 0.90 1.5	0.33 0.87 1.5	C C C	C C C	ND1	1	
AP-202-TA1	10.0-24.0	CL-SC	115.5	11.1	50 180 380	0.35 0.88 1.4	0.35 0.88 1.4	C C C	C C C	ND1	1	
	24.0-35.0	CL-SC	125.8	9.1	50 180 380	0.22 0.90 1.5	0.35 0.90 1.5	C C C	C C C	ND1	1	
AP-203-TA1	6.0-40.0	SC-CL	119.5	11.7	50 180 380	0.27 0.63 1.2	0.27 0.63 1.3	C C C	C C C	ND1	1	
AP-204-TA1	8.0-15.0	SC	126.5	7.8	50 180 380	0.20 1.2 1.9	0.48 1.2 1.9	C C C	C C C	ND2	1	- No. 30 soil
AP-205-TA1	11.0-40.0	SC	120.4	8.7	50 180 380	0.25 0.90 1.4	0.37 0.90 1.4	C C C	C C C	ND1	1	Holes repunched to start flow at 50 mm head
AP-207-TA1	0.0-9.0	SC	120.4	9.0	50 180 380	0.25 1.0	0.43 1.0	C C	C C	ND2	1	* * * could be ND1 based on clear flow and final 1.0 mm pinhole diameter
	9.0-18.0	SC	112.7	12.1	50 180 380	0.63	1.1	CL	SC	D1	3	- No. 30 soil
	18.0-26.0	SC	121.0	9.9	50 180 380	0.35	1.1	SC	SC	D2	3	
	26.0-40.0	SC	122.0	9.9	50 180 380	0.17	1.1	SC	SC	D2	3	
AP-208-TA1	11.0-19.0	SC	114.8	11.1	50 180 380	0.32 0.82 1.4	0.33 0.82 1.4	C C C	C C C	ND1	1	
	19.0-40.0	SC	115.7	10.4	50 180 380	0.28 1.3	0.50 1.3	SC SC	C C	ND3	1	Hole repunched at 2 min at 50 mm head

APPENDIX G - Continued
TUCSON AQUEDUCT - REACH 1 - BORROW AREA

Sample identification		Unified soil class.	Pinhole test data summary: Placement d (lb/ft³)	w (%)	Head (mm)	Q_i (mL/s)	Q_f (mL/s)	Turbidity* Initial	Final	Classification	Crumb test	Remarks
AP-209-TA1	12.0-19.0	SC-SM	125.7	9.2	50	0.27	0.27	C	C	ND1	1	
					180	0.75	0.75	C	C			
					380	1.2	1.2	C	C			
	19.0-31.0	SC	118.7	9.9	50	0.55	0.55	C	C	ND3	1	Could be ND1 based on final pinhole size and clear flow at 180 mm head
					180	1.2	1.2	C	C			
					380							
	31.0-40.0	SC	120.4	9.0	50	0.35	0.65	SC	SC	ND3	3	Hole repunched three times. Questionable flow rate data
					180	0.77	1.1	SC	SC			
					380							
AP-210-TA1	34.0-40.0	SC	122.5	10.0	50	0.22	0.35	C	C	ND1	1	
					180	0.82	0.82	C	C			
					380	1.3	1.3	C	C			
AP-212-TA1	0.0-9.0	SC	123.1	9.2	50	0.27	0.42	C	C	ND3	1	Should be ND1 based on final pinhole sizes and clear flow
					180	1.1	1.1	C	C			
					380	1.6	1.6	C	C			
	21.0-34.0	SC-CL	121.5	10.6	50	0.35	1.1	SC	SC	D2	1	
					180							
					380							
AP-213-TA1	9.0-26.0	SC-CL	118.5	10.5	50	0.30	0.37	SC	C	ND1	1	
					180	0.85	0.83	C	C			
					380	1.3	1.3	C	C			
	26.0-40.0	SC-CL	111.9	14.2	50	0.33	0.33	C	C	ND1	1	
					180	0.88	0.88	C	C			
					380	1.4	1.4	C	C			
AP-214-TA1	9.0-23.0	SC-CL	121.1	8.6	50	0.22	0.32	C	C	ND1	1	
					180	0.83	0.83	C	C			
					380	1.4	1.4	C	C			
	32.0-40.0	SC	118.6	9.4	50	0.38	0.68	SC	C	ND3	1	
					180	1.6	1.6	C	C			
					380							
AP-215-TA1	23.0-31.0	SC	116.3	10.6	50	0.32	0.58	SC	SC	ND3	1	
					180	1.2	1.9	SC	SC			
					380							
	31.0-40.0	SC	118.5	10.0	50	0.30	1.1	SC	SC	D2	2	
					180							
					380							
AP-216-TA1	5.0-10.0	SC	122.4	9.8	50	0.30	0.30	C	C	ND1	1	
					180	0.76	0.78	C	C			
					380	1.3	1.3	C	C			
	15.0-20.0	SC	121.2	9.2	50	0.30	0.30	C	C	ND1	1	- No. 30 soil
					180	0.82	0.82	C	C			
					380	1.4	1.4	C	C			
AP-216-TA1	20.0-25.0	SC-CL	117.3	11.9	50	0.33	0.35	C	C	ND1	1	- No. 30 soil
					180	0.90	0.90	C	C			
					380	1.5	1.5	C	C			
	25.0-30.0	SC-CL	117.7	11.3	50	0.27	0.30	C	C	ND1	1	Repunched hole to start flow at 50 mm head - No. 30 soil
					180	0.78	0.78	C	C			
					380	1.3	1.3	C	C			
	30.0-35.0	SC	116.0	10.3	50	0.33	0.33	C	C	ND1	1	- No. 30 soil
					180	0.82	0.82	C	C			
					380	1.3	1.3	C	C			
	35.0-40.0	SC-CL	117.9	11.5	50	0.32	1.1	SC	SC	D1	3	- No. 30 soil
					180							
					380							
	40.0-45.0	SC	117.2	11.3	50	0.30	1.1	SC	SC	D2	2	- No. 30 soil
					180							
					380							
	45.0-50.0	SC	118.8	9.3	50	0.32	1.1	SC	SC	D1	2	- No. 30 soil
					180							
					380							
AP-217-TA1	5.0-10.0	SC	117.5	11.2	50	0.38	1.1	SC	SC	D1	3	- No. 30 soil
					180							
					380							
	10.0-15.0	SC	121.0	9.0	50	0.32	0.32	C	C	ND1	1	- No. 30 soil
					180	0.85	0.85	C	C			
					380	1.5	1.5	C	C			
	15.0-20.0	SC	121.6	9.4	50	0.35	0.35	C	C	ND1	1	- No. 30 soil
					180	0.83	0.83	C	C			
					380	1.4	1.4	C	C			
	20.0-25.0	SC	117.9	10.1	50	0.35	0.35	C	C	ND1	2	- No. 30 soil
					180	0.83	0.83	C	C			
					380	1.4	1.4	C	C			
	25.0-30.0	SC	117.6	11.0	50	0.35	0.35	C	C	ND1	1	- No. 30 soil
					180	0.88	0.88	C	C			
					380	1.4	1.4	C	C			

APPENDIX G - Continued
TUCSON AQUEDUCT - REACH 1 - BORROW AREA

Sample identification		Unified soil class.	Placement d (lb/ft³)	w (%)	Pinhole test data summary Head (cm)	Q_i (mL/s)	Q_f (mL/s)	Turbidity* Initial	Final	Classification	Crumb test	Remarks
	30.0-35.0	SC-CL	108.7	14.4	50	0.35	0.43	SC	C	ND1	1	- No. 30 soil
					180	0.93	0.93	C	C			
					380	1.6	1.6	C	C			
	35.0-40.0	SC	124.4	8.3	50	0.32	1.1	SC	SC	D1	3	- No. 30 soil
					180							
					380							
AP-217-TA1	40.0-45.0	SC-CL	114.7	11.9	50	0.38	0.95	CL	C	ND4	1	
					180	2.1	2.1	C	C			
					380							
	45.0-50.0	SC	117.6	11.4	50	0.38	0.52	SC	C	ND2	1	- No. 30 soil
					180	1.2	1.2	C	C			
					380	1.9	1.9	C	C			
AP-218-TA1	5.0-10.0	SC	119.1	9.8	50	0.25	0.30	C	C	ND1	1	- No. 30 soil
					180	0.80	0.80	C	C			
					380	1.3	1.3	C	C			
	10.0-15.0	SC	122.2	8.6	50	0.30	0.32	C	C	ND1	1	- No. 30 soil
					180	0.80	0.80	C	C			
					380	1.4	1.4	C	C			
	15.0-20.0	SC	118.8	10.5	50	0.42	1.1	SC	SC	D1	1	- No. 30 soil
					180							
					380							
	20.0-25.0	SC	119.3	10.1	50	0.28	1.0	SC	SC	D2	1	- No. 30 soil
					180							
					380							
	25.0-30.0	SC	123.9	8.3	50	0.42	1.1	CL	SC	D1	3	- No. 30 soil
					180							
					380							
	30.0-35.0	CL	111.6	11.9	50	0.42	1.1	CL	CL	D1	1	- No. 30 soil
					180							
					380							
	35.0-40.0	SC	124.6	8.7	50	0.37	1.1	CL	SC	D1	3	- No. 30 soil
					180							
					380							
	40.0-45.0	SC	122.2	9.2	50	0.42	1.2	CL	CL	D1	2	- No. 30 soil
					180							
					380							
	45.0-50.0	SC	112.1	10.4	50	0.30	0.70	CL	C	ND3	1	
					180	1.7	1.7	C	C			
					380							
AP-219-TA1	0.0-5.0	SC-SM	126.6	7.1	50	0.25	0.38	C	C	ND2	1	- No. 30 soil
					180	0.95	0.95	C	C			
					380	1.6	1.7	C	C			
	5.0-10.0	SC	120.8	9.2	50	0.28	0.32	C	C	ND1	1	- No. 30 soil
					180	0.80	0.80	C	C			
					380	1.3	1.3	C	C			
AP-219-TA1	10.0-15.0	SC	123.2	7.5	50	0.33	0.35	SC	C	ND1	2	- No. 30 soil
					180	0.92	0.92	C	C			
					380	1.4	1.4	C	C			
	15.0-20.0	SC	120.2	9.7	50	0.33	1.1	SC	CL	D1	2	- No. 30 soil
					180							
					380							
	20.0-25.0	SC	117.8	9.9	50	0.33	1.0	CL	SC	D1	2	- No. 30 soil
					180							
					380							
	25.0-30.0	SC-CL	115.9	11.7	50	0.43	1.2	CL	CL	D1	1	- No. 30 soil
					180							
					380							
	30.0-35.0	SC-CL	112.6	12.8	50	0.27	1.2	CL	SC	D1	2	- No. 30 soil
					180							
					380							
	35.0-40.0	SC	116.9	9.3	50	0.28	1.1	CL	SC	D1	3	- No. 30 soil
					180							
					380							
	40.0-45.0	SC	111.8	11.0	50	0.30	1.1	CL	SC	D1	1	- No 30 soil
					180							
					380							
	45.0-50.0	SC	116.4	9.6	50	0.25	1.1	CL	SC	D1	3	- No. 30 soil
					180							
					380							

* D = Dark/distinctly cloudy CL = Cloudy SC = Slightly cloudy C = Clear

APPENDIX H
MISC. SPECIMENS NO. 4 and 10 COMPARISON TESTS

Sample identification		Unified soil class.	Placement d (lb/ft³)	Placement w (%)	Pinhole test data summary: Head (mm)	Q_i (mL/s)	Q_f (mL/s)	Turbidity* Initial	Turbidity* Final	Classification	Crumb test	Remarks
LaVerkin Springs	61Z-1	MH	94.0	25.3	50 180 380	0.37	1.1	SC	SC	ND1	1	- No. 4 soil
	61Z-1	MH	93.1	25.3	50 180 380	0.33	1.0	SC-CL	SC	ND1	-	- No. 10 soil - replicate pinhole test
	61Z-2	MH-ML	98.5	22.3	50 180 380	0.37 2.1	0.25 1.2	SC SC	SC SC	ND4	1	- No. 4 soil
	61Z-2	MH-ML		22.3	50 180 380	0.18 1.1	0.43 2.4	SC SC	SC SC	ND4	-	- No. 10 soil - replicate pinhole test
	61Z-3	MH	94.9	27.0	50 180 380	0.38	0.68	SC	SC	ND4	2	Test stopped at 50 mm ND due to excessive plugging. - No. 4 soil
	61Z-3	MH	94.0	27.0	50 180 380	0.20	0.70	SC	SC	ND4	-	- No. 10 soil - test stopped at 50 mm head due to excessive plugging
San Justo Dam	59M-396	CL	106.9	17.8	50 180 380	0.10 0.60 2.2	0.11 1.0 2.7	C SC SC	C SC C-SC	ND3	-	- No. 10 soil
	59M-398	SC	105.5	18.9	50 180 380	0.23	1.1	SC	SC	D2	-	- No. 10 soil
	59M-404	CL	101.9	15.8	50 180 380	0.38 2.4	0.87 2.5	CL D	CL D	ND4	-	- No. 4 soil
	59M-404	CL	103.8	15.8	50 180 380	0.13	1.1	CL	D	D2	-	Replicate test using - No. 10 soil
	59M-404	CL	103.8	15.8	50 180 380	0.21	1.1	CL	D	D2	-	Replicate test using - No. 10 soil
	59M-405	CL	105.8	16.3	50 180 380	0.16 2.6	0.88 2.5	CL D	CL D	ND4	-	- No. 10 soil - Hole repunched to start flow at 50 mm head
	59M-405 (w/screen O/S)	CL	106.3	16.3	50 180 380	0.10	0.92	D	D	ND4	-	- No. 10 soil - Hole repunched to start flow at 50 mm head
	59M-406	CL	102.4	20.2	50 180 380	0.14	1.0	CL	D	D2	-	- No. 10 soil
	59M-410	CL	98.1	25.7	50 180 380	0.15 1.7	0.67 1.6	C C	C C	ND3	-	- No. 10 soil - Hole repunched to start flow at 50 mm head
	59M-414 (w/screen O/S)	SC	107.2	18.0	50 180 380	0.18 0.82 1.4	0.20 0.82 1.4	C C C	C C C	ND1	-	
	59M-414 (w/screen O/S)	SC	107.8	18.0	50 180 380	0.27 0.57 0.95	0.18 0.55 0.92	C C C	C C C	ND1	-	

* D = Dark/distinctly cloudy CL = Cloudy SC = Slightly cloudy C = Clear

APPENDIX I
MCGEE CREEK DAM - LIME-TREATED SOIL IN BORROW AREA F

Sample identification	Unified soil class.	Placement d (kg/m³)	Placement w (%)	Pinhole test data summary Head (mm)	Q_1 (mL/s)	Q_2 (mL/s)	Turbidity* Initial	Turbidity* Final	Classification	Crumb test	Remarks
BAF: F-2 North End	CL-ML	1825	10.7	50	0.4	0.4	C	C	ND1	1	Natural soil
				180	0.8	0.9	C	C			
				380	1.4	1.4	C	C			
	CL-ML	1829	10.7	50	0.4	0.4	C	C	ND1	1	Natural soil replicate test
				180	0.8	0.9	C	C			
				380	1.4	1.5	C	C			
	SC	1996	17.4	50	0.2	0.2	C	C	ND1	1	Premix
				180	0.7	0.7	C	C			
				380	1.1	1.2	C	C			
	SC	1967	17.4	50	0.2	0.2	C	C	ND1	1	Premix replicate test
				180	0.8	0.8	C	C			
				380	1.3	1.3	C	C			
	SM	1765	15.9	50	0.5	0.5	C	C	ND1	1	Final mix
				180	0.9	0.9	C	C			
				380	1.8	1.5	C	C			
	SM	1787	15.9	50	0.5	0.5	C	C	ND1	1	Final mix replicate test
				180	1.0	1.1	C	C			
				380	1.6	1.6	C	C			
BAF: F-2 South end	CL-ML	1902	8.2	50	0.3	0.3	C	C	ND1	1	Natural soil
				180	0.9	0.9	C	C			
				380	1.5	1.5	C	C			
	CL-ML	1779	8.2	50	0.5	0.5	C	C	ND2	1	Natural soil replicate test
				180	1.1	1.2	C	C			
				380	1.8	1.9	C	C			
	ML	1830	17.1	50	0.5	0.5	C	C	ND3	1	Premix
				190	1.2	1.2	C	C			
				380	2.0	2.1	C	C			
	ML	1838	17.1	50	0.3	0.3	C	C	ND1	1	Premix replicate test
				180	0.7	0.7	C	C			
				380	1.1	1.1	C	C			
	CL-ML	1857	13.3	50	0.5	0.5	C	C	ND3	1	Final mix
				180	1.3	1.3	C	C			
				380	2.0	2.1	C	C			
	CL-ML	1902	13.3	50	0.6	0.7	C	C	ND3	1	Final mix replicate test
				180	1.3	1.3	C	C			
				380	2.1	2.2	C	C			
BAF: F-3 Northwest corner	CL-ML	1781	16.0	50	0.3	0.3	C	C	ND1	1	Natural soil
				180	0.6	0.6	C	C			
				380	1.0	1.1	C	C			
	CL-ML	1786	16.0	50	0.5	0.5	C	C	ND2	1	Natural soil replicate test
				180	1.1	1.2	C	C			
				380	1.7	1.8	C	C			
	SC-CL	1513	21.0	50	0.3	0.3	C	C	ND1	1	Premix
				180	0.7	0.8	C	C			
				380	1.2	1.3	C	C			
	SC-CL	1512	21.0	50	0.2	0.3	C	C	ND1	1	Premix replicate test
				180	0.5	0.8	C	C			
				380	1.1	1.3	C	C			
	CL-ML	1826	18.4	50	0.3	0.3	C	C	ND1	1	Final mix
				180	0.7	0.9	C	C			
				380	1.3	1.3	C	C			
	CL-ML	1798	18.4	50	0.3	0.3	C	C	ND1	1	Final mix replicate test
				180	0.7	0.7	C	C			
				380	1.1	1.2	C	C			
BAF: F-3 Southwest corner	CL-ML	1788	12.9	50	0.5	0.4	C	C	ND1	1	Natural soil
				180	0.8	1.0	C	C			
				380	1.4	1.6	C	C			
	CL-ML	1794	12.9	50	0.4	0.4	C	C	ND1	1	Natural soil replicate test
				180	0.9	1.0	C	C			
				380	1.5	1.6	C	C			
	SC-SM	1680	15.5	50	0.4	0.3	C	C	ND1	1	Premix
				180	0.8	0.8	C	C			
				380	1.3	1.3	C	C			
	SC-SM	1679	15.5	50	0.4	0.4	C	C	ND1	1	Premix replicate test
				180	0.7	0.9	C	C			
				380	1.3	1.4	C	C			
	SC-SM	1741	16.6	50	0.3	0.3	C	C	ND2	1	Final mix
				180	1.0	1.0	C	C			
				360	2.0	2.0	C	C			
	SC-SM	1745	16.6	50	0.3	0.3	C	C	ND1	1	Final mix replicate test
				180	0.7	0.7	C	C			
				380	1.2	1.2	C	C			
BAF: F-4 North end	CL	1794	13.5	50	0.4	1.1	CL	CL	D1	3	Natural soil
				180							
				380							

APPENDIX I - Continued
MCGEE CREEK DAM - LIME-TREATED SOIL IN BORROW AREA F

Sample identification	Unified soil class.	Pinhole test data summary: Placement d (kg/m³)	w (%)	Head (mm)	Qi (mL/s)	Qr (mL/s)	Turbidity* Initial	Final	Classification	Crumb test	Remarks
	CL	1904	13.5	50 180 380	0.6	1.1	CL	CL	D1	3	Natural soil replicate test
	CL	1563	19.1	50 180 380	0.3 0.7 1.2	0.3 0.8 1.2	C C C	C C C	ND1	1	Premix
	CL	1608	19.1	50 180 380	0.3 0.8 1.3	0.3 0.8 1.3	C C C	C C C	ND1	1	Premix replicate test
	CL	1750	17.3	50 180 380	0.5 1.1 1.7	0.5 1.1 1.8	C C C	C C C	ND2	1	Final mix
	CL	1751	17.3	50 180 380	0.2 1.0 1.6	0.1 1.0 1.6	C C C	C C C	ND1	1	Final mix replicate test
BAF: F-4 South end	CL	1934	9.3	50 180 380	0.5	1.1	CL	CL	D1	4	Natural soil
	CL	1937	9.3	50 180 380	0.4	1.1	CL	CL	D1	4	Natural soil replicate test
	CL	1743	12.0	50 180 380	0.3 0.7 1.1	0.3 0.7 1.1	C C C	C C C	ND1	1	Premix
	CL	1750	12.0	50 180 380	0.3 0.8 1.2	0.3 0.8 1.3	C C C	C C C	ND1	1	Premix replicate test
	SC	1696	17.5	50 180 380	0.4 0.9 1.9	0.4 1.0 2.0	C C C	C C C	ND2	1	Final mix replicate test
	SC	1666	17.5	50 180 380	0.4 0.9 2.0	0.4 1.0 2.0	C C C	C C C	ND2	1	Final mix replicate test
BAF: F-5 Southwest corner	CL-ML	1724	8.5	50 180 380	0.4 1.0 1.6	0.4 1.0 2.3	C SC CL	C SC CL	ND2	3	Natural soil
	CL-ML	1724	8.5	50 180 380	0.5 1.2 2.2	0.5 1.5 2.5	C SC CL	C CL CL	ND3	3	Natural soil replicate test
	CL-ML	1735	12.4	50 180 380	0.4 0.9 1.3	0.3 0.9 1.4	C C C	C C C	ND1	1	Premix
	CL-ML	1711	12.4	50 180 380	0.4 0.9 1.4	0.4 0.9 1.4	C C C	C C C	ND1	1	Premix replicate test
	SC	1695	19.2	50 180 380	0.4 1.0 1.5	0.4 1.0 1.5	C C C	C C C	ND1	1	Final mix
	SC	1705	19.2	50 180 380	0.4 0.9 1.6	0.4 1.0 1.6	C C C	C C C	ND1	1	Final mix replicate test
BAF: F-5 Northeast corner	SC-CL	1865	13.5	50 180 380	0.4 0.8 1.3	0.3 0.8 1.3	C C C	C C C	ND1	3	Natural soil
	SC-CL	1879	13.5	50 180 380	0.3 0.8 1.3	0.3 0.8 1.3	C C C	C C C	ND1	3	Natural soil replicate test
	SM	1524	23.7	50 180 380	0.5 1.1 1.7	0.5 1.1 1.7	C C C	C C C	ND2	1	Premix
	SM	1526	23.7	50 180 380	0.4 1.0 1.7	0.4 1.0 1.7	C C C	C C C	ND2	1	Premix replicate test
	SC-SM	1725	17.9	50 180 380	0.5 1.1 1.7	0.5 1.1 1.8	C C C	C C C	ND2	1	Final mix
	SC-SM	1723	17.9	50 180 380	0.5 1.0 1.6	0.4 1.0 1.6	C C C	C C C	ND1	1	Final mix replicate test
BAF: F-6 Northwest corner	CL	1524	12.7	50 180 380	0.3 0.6 1.0	0.3 0.7 1.1	C C C	C C C	ND1	3	Natural soil
	CL	1511	12.7	50 180 380	0.3 0.5 1.0	0.3 0.5 0.9	C C C	C C C	ND1	3	Natural soil replicate test

APPENDIX I - Continued
MCGEE CREEK DAM - LIME-TREATED SOIL IN BORROW AREA F

Sample identification	Unified soil class.	Placement d (kg/m³)	Placement w (%)	Pinhole test data summary: Head (mm)	Q_i (mL/s)	Q_f (mL/s)	Turbidity[a] Initial	Turbidity[a] Final	Classification	Crumb test	Remarks
	SC-CL	1410	18.8	50 180 380	0.3 0.8 1.3	0.3 0.8 1.3	C C C	C C C	ND1	1	Premix
	SC-CL	1412	18.8	50 180 380	0.2 0.7 1.3	0.2 0.8 1.3	C C C	C C C	ND1	1	Premix replicate test
	SC	1627	21.6	50 180 380	0.3 0.6 1.0	0.3 0.6 1.0	C C C	C C C	ND1	1	Final mix
	SC	1673	21.6	50 180 380	0.4 1.0 1.9	0.4 1.2 2.0	C C C	C C C	ND2	1	Final mix replicate test
BAF: F-6 Southeast corner	CL	1744	10.8	50 180 380	0.3 0.6 1.0	0.3 0.6 1.0	C C C	C C C	ND1	1	Natural soil
	CL	1736	10.8	50 180 380	0.5 0.9 1.5	0.4 0.9 1.5	C C C	C C C	ND1	1	Natural soil replicate test
	SC-CL	1520	21.4	50 180 380	0.3 0.8 1.3	0.3 0.8 1.3	C C C	C C C	ND1	1	Premix
	SC-CL	1525	21.4	50 180 380	0.3 0.7 1.2	0.3 0.7 1.2	C C C	C C C	ND1	1	Premix replicate test test
	CL	1694	19.9	50 180 380	0.4 0.8 1.3	0.4 0.9 1.4	C C C	C C C	ND1	1	Final mix
	CL	1686	19.9	50 180 380	0.4 0.8 1.3	0.4 0.9 1.4	C C C	C C C	ND1	1	Final mix replicate test
BAF: F-7 Center	CL-ML	1849	10.1	50 180 380	0.3 0.7 1.3	0.4 0.8 1.3	C C C	C C C	ND1	1	Natural soil
	CL-ML	1884	10.1	50 180 380	0.2 0.5 0.9	0.2 0.6 0.9	C C C	C C C	ND1	1	Natural soil replicate test
	SC-SM	1779	16.7	50 180 380	0.6 1.4 2.7	0.5 1.5 2.4	C C C	C C C	ND3	1	Premix
	SC-SM	1817	16.7	50 180 380	0.5 1.1 1.9	0.5 1.1 2.0	C C C	C C C	ND2	1	Premix replicate test
	SC-SM	1855	13.7	50 180 380	0.3 0.8 1.2	0.3 0.8 1.2	C C C	C C C	ND1	1	Final mix
	SC-SM	1846	13.7	50 180 380	0.3 0.9 1.2	0.4 0.9 1.3	C C C	C C C	ND1	1	Final mix replicate test
BAF: F-8 Southeast corner	CL	1765	15.8	50 180 380	0.3 2.0	0.8 2.4	SC	SC	ND4	3	Natural soil
	CL	1765	15.8	50 180 380	0.3 2.0	0.9 2.4	SC SC	SC SC	ND4	3	Natural soil replicate test
	CL	1620	17.1	50 180 380	0.2 0.6 1.0	0.2 0.6 1.1	C C C	C C C	ND1	1	Premix
	CL	1628	17.1	50 180 380	0.4 0.8 1.3	0.4 0.9 1.3	C C C	C C C	ND1	1	Premix replicate test
	SC	1472	26.3	50 180 380	0.4 0.7 1.2	0.4 0.8 1.3	C C C	C C C	ND1	1	Final mix
	SC	1486	26.3	50 180 380	0.3 0.5 0.9	0.3 0.7 1.2	C C C	C C C	ND1	1	Final mix replicate test
BAF: F-8 Northwest corner	CL	1776	13.5	50 180 380	0.4	1.2	CL	CL	D1	4	Natural soil
	CL	1780	13.5	50 180 380	0.4	1.2	CL	CL	D1	4	Natural soil replicate test
	SC	1510	22.4	50 180 380	0.2 0.4 1.1	0.2 0.4 1.1	C C C	C C C	ND1	1	Premix

APPENDIX I - Continued
MCGEE CREEK DAM - LIME-TREATED SOIL IN BORROW AREA F

Sample identification	Unified soil class.	Placement d (kg/m³)	Placement w (%)	Pinhole test data summary Head (mm)	Q_i (mL/s)	Q_f (mL/s)	Turbidity Initial	Turbidity Final	Classification	Crumb test	Remarks
	SC	1507	22.4	50 180 380	0.2 0.7 1.1	0.2 0.7 1.2	C C C	C C C	ND1	1	Premix replicate test
	CL	1619	20.3	50 180 380	0.3 0.8 1.3	0.3 0.8 1.2	C C C	C C C	ND1	1	Final
	CL	1685	20.3	50 180 380	0.3 0.7 1.2	0.3 0.6 1.1	C C C	C C C	ND1	1	Final mix replicate test
BAF: F-9 Northwest corner	CL	1970	10.5	50 180 380	0.6	1.0	SC	SC	D2	3	Natural soil
	CL	1971	10.5	50 180 380	0.5	1.0	SC	SC	D2	3	Natural soil replicate test
	CL	1311	24.5	50 180 380	0.4 1.0 1.5	0.4 1.0 1.6	C C C	C C C	ND1	1	Premix
	CL	1313	24.5	50 180 380	0.2 0.7 1.1	0.2 0.7 1.1	C C C	C C C	ND1	1	Premix replicate test
	CL-ML	1705	16.9	50 180 380	0.3 0.8 1.4	0.3 0.9 1.4	C C C	C C C	ND1	1	Final mix
		1705	16.9	50 180 380	0.5 0.9 1.5	0.5 1.0 1.5	C C C	C C C	ND1	-	Final mix
BAF: F-9 Southeast corner	SC	1770	13.6	50 180 380	0.3	1.0	SC	SC	D2	3	Natural soil
	SC	1746	13.6	50 180 380	0.3	1.3	SC	SC	D2	3	Natural soil replicate test
	SM	1627	19.8	50 180 380	0.5 1.1 1.8	0.5 1.1 1.8	C C C	C C C	ND1	1	Final mix
	SM	1627	19.8	50 180 380	0.7 1.1 1.8	0.5 1.1 1.8	C C C	C C C	ND1	1	Final mix replicate test
BAF: F-10 Southeast corner	CL	1894	11.8	50 180 380	0.2 1.6	0.4 2.3	SC SC	SC SC	ND4	1	Natural soil
	CL	1904	11.8	50 180 380	0.3 0.8 1.3	0.3 0.9 1.3	C C C	C C C	ND1	1	Natural soil replicate test
	SM-SC	1612	22.9	50 180 380	0.4 0.8 1.2	0.4 0.8 1.2	C C C	C C C	ND1	1	Premix
	SM-SC	1567	22.9	50 180 380	0.3 0.6 1.1	0.3 0.6 1.1	C C C	C C C	ND1	1	Premix replicate test
	SC	1517	21.2	50 180 380	0.3 0.7 1.2	0.3 0.7 1.2	C C C	C C C	ND1	1	Final mix
	SC	1497	21.2	50 180 380	0.3 0.6 1.0	0.3 0.6 1.0	C C C	C C C	ND1	1	Final mix replicate test
BAF: F-10 Northwest corner	CL	1736	16.3	50 180 380	0.4 1.1 1.6	0.6 1.1 1.6	C C C	C C C	ND1	3	Natural soil
	CL	1726	16.3	50 180 380	0.5 0.8	0.5 1.5			Test stopped not classified (prob. ND4)		Natural soil replicate test
	SC-SM	1613	19.8	50 180 380	0.5 1.0 1.6	0.5 1.0 1.6	C C C	C C C	ND1	1	Premix
	SC-SM	1605	19.8	50 180 380	0.4 0.9 1.3	0.4 0.9 1.3	C C C	C C C	ND1	1	Premix replicate data
	SC	1667	17.4	50 180 380	0.3 0.7 1.1	0.3 0.7 1.1	C C C	C C C	ND1	1	Final mix
	SC-CL	1640	17.4	50 180 380	0.3 0.7 1.2	0.3 0.7 1.2	C C C	C C C	ND1	1	Final mix replicate data

APPENDIX I - Continued
MCGEE CREEK DAM - LIME-TREATED SOIL IN BORROW AREA F

Sample identification	Unified soil class.	Pinhole test data summary: Placement d (kg/m³)	Placement w (%)	Head (mm)	Q_i (mL/s)	Q_f (mL/s)	Turbidity[a] Initial	Turbidity[a] Final	Classification	Crumb test	Remarks
BAF: F-10 Center	CL	1726	21.0	50	0.4	0.4	C	C	ND1	1	Natural soil
				180	0.8	0.9	C	C			
				380	1.3	1.3	C	C			
	CL	1712	21.0	50	0.3	0.3	C	C	ND1	1	Natural soil - replicate test
				180	0.7	0.8	C	C			
				380	1.3	1.3	C	C			
	CL	1668	18.7	50	0.5	0.5	C	C	ND1	1	Premix
				180	1.0	1.0	C	C			
				380	1.7	1.7	C	C			
	CL	1641	18.7	50	0.4	0.3	C	C	ND1	1	Premix replicate test
				180	0.6	0.8	C	C			
				380	1.1	1.2	C	C			
	CL	1672	21.5	50	0.2	0.2	C	C	ND1	1	Final mix
				180	0.7	0.7	C	C			
				380	1.2	1.2	C	C			
	CL	1672	21.5	50	0.1	0.1	C	C	ND1	1	Final mix replicate test
				180	0.3	0.3	C	C			
				380	1.0	1.0	C	C			
BAF: F-11 Northwest corner	CL-ML	1864	9.2	50	0.3	0.3	C	C	ND1	3	Natural soil
				180	0.5	0.8	C	C			
				380	1.0	1.2	C	C			
	CL-ML	1813	9.2	50	0.4	0.4	C	C	ND1	3	Natural soil replicate test
				180	0.9	0.9	C	C			
				380	1.5	1.5	C	C			
	SC	1859	12.9	50	0.3	0.3	C	C	ND1	1	Premix
				180	0.8	0.7	C	C			
				380	1.2	1.2	C	C			
	SC	1850	12.9	50	0.5	0.5	C	C	ND1	1	Premix replicate test
				180	0.9	0.9	C	C			
				380	1.2	1.2	C	C			
	SM-SC	1798	15.2	50	0.3	0.3	C	C	ND1	1	Final mix
				180	0.8	0.8	C	C			
				380	1.2	1.2	C	C			
	SM-SC	1796	15.2	50	0.3	0.3	C	C	ND1	1	Final mix replicate test
				180	0.8	0.8	C	C			
				380	1.2	1.2	C	C			
BAF: F-11 Southeast corner	CL-ML	1911	14.5	50	0.6	0.6	C	C	ND2	1	Natural soil
				180	1.2	1.3	C	C			
				380	1.9	2.0	C	C			
	CL-ML	1933	14.5	50	0.4	0.4	C	C	ND2	1	Natural soil replicate test
				180	1.1	1.1	C	C			
				380	1.8	1.8	C	C			
	SM-SC	1725	18.1	50	0.3	0.3	C	C	ND1	1	Premix
				180	0.7	0.7	C	C			
				380	1.1	1.2	C	C			
	SM-SC	1739	18.1	50	0.1	0.3	C	C	ND1	1	Premix replicate test
				180	0.7	0.7	C	C			
				380	1.2	1.2	C	C			
	SM	1758	16.6	50	0.4	0.4	C	C	ND1	1	Final mix
				180	0.8	0.8	C	C			
				380	1.4	1.4	C	C			
	SM	1738	16.6	50	0.4	0.4	C	C	ND1	1	Final mix replicate test
				180	0.9	0.9	C	C			
				380	1.4	1.4	C	C			
BAF: F-11B Southeast corner	CL	1816	16.6	50	0.3	0.3	C	C	ND1	1	Natural soil
				180	0.7	0.7	C	C			
				380	1.1	1.1	C	C			
	CL	1819	16.6	50	0.2	0.3	C	C	ND1	1	Natural soil replicate test
				180	0.7	0.7	C	C			
				380	1.2	1.2	C	C			
	SC	1517	21.6	50	0.3	0.3	C	C	ND1	1	Premix
				180	0.8	0.8	C	C			
				380	1.3	1.3	C	C			
	SC	1562	21.6	50	0.2	0.3	C	C	ND1	1	Premix replicate test
				180	0.7	0.7	C	C			
				380	1.2	1.2	C	C			
BAF: F-11B Northwest corner	CL	1718	19.0	50	0.2	0.3	C	C	ND1	2	Natural soil
				180	0.6	0.7	C	C			
				380	0.9	1.0	C	C			
	CL	1712	19.0	50	0.2	0.4	C	C	ND2	2	Natural soil replicate test
				180	1.1	1.1	C	C			
				380	1.8	1.8	C	C			
	SC-CL	1685	20.1	50	0.3	0.3	C	C	ND1	1	Premix
				180	0.8	0.8	C	C			
				380	1.3	1.3	C	C			

APPENDIX I - Continued
MCGEE CREEK DAM - LIME-TREATED SOIL IN BORROW AREA F

Sample identification	Unified soil class.	Placement d (kg/m³)	Placement w (%)	Pinhole test data summary: Head (mm)	Qi (mL/s)	Qf (mL/s)	Turbidity[a] Initial	Turbidity[a] Final	Classification	Crumb test	Remarks
	SC-CL	1685	20.1	50 180 380	0.3 0.8 1.3	0.3 0.8 1.3			ND1	1	Premix replicate test
BAF: F-12 Southeast corner	CL	1841	13.0	50 180 380	0.2	1.0	CL	CL	D2	4	Natural soil
	CL	1793	13.0	50 180 380	0.2	1.0	CL	CL	D2	4	Natural soil replicate test
	SC	1528	19.2	50 180 380	0.3 0.7 1.1	0.3 0.7 1.1	C C C	C C C	ND1	1	Premix
	SC	1545	19.2	50 180 380	0.4 0.9 1.4	0.4 0.9 1.4	C C C	C C C	ND1	1	Premix replicate test
	SC	1523	28.5	50 180 380	0.4 0.7 1.4	0.4 0.9 1.4	C C C	C C C	ND1	1	Final mix
	SC	1533	28.5	50 180 380	0.3 0.6 1.1	0.3 0.7 1.1	C C C	C C C	ND1	1	Final mix replicate test
BAF: F-12 Northwest corner	CL	1839	16.7	50 180 380	0.2 2.1	0.8 2.5	SC SC	SC SC	ND4	4	Natural soil
	CL	1837	16.7	50 180 380	0.2 2.0	0.6 2.6	SC SC	SC SC	ND4	4	Natural soil replicate test
	SC	1647	21.4	50 180 380	0.3 0.7 1.1	0.3 0.7 1.2	C C C	C C C	ND1	1	Premix
	SC	1647	21.4	50 180 380	0.3 0.7 1.2	0.3 0.7 1.2	C C C	C C C	ND1	1	Premix replicate test
BAF: F-13 Southeast corner	SC-CL	1623	13.3	50 180 380	0.4 0.8 1.4	0.4 0.9 1.4	C C C	C C C	ND1	1	Natural soil
	SC-CL	1631	13.3	50 180 380	0.3 0.7 1.2	0.3 0.8 1.2	C C C	C C C	ND1	1	Natural soil replicate test
	SC-CL	1500	15.3	50 180 380	0.2 0.7 1.1	0.3 0.7 1.1	C C C	C C C	ND1	1	Premix
	SC-CL	1606	15.3	50 180 380	0.3 0.6 1.0	0.3 0.7 1.2	C C C	C C C	ND1	1	Premix replicate test
	SC	1441	27.3	50 180 380	0.3 0.7 1.2	0.3 0.7 1.2	C C C	C C C	ND1	1	Final mix
	SC	1442	27.3	50 180 380	0.4 0.8 1.4	0.3 0.9 1.4	C C C	C C C	ND1	1	Final mix replicate test
BAF: F-13 Northwest corner	CL	1758	13.6	50 180 380	0.4 0.8 1.4	0.4 0.9 1.5	C C C	C C C	ND1	1	Natural soil
	CL	1730	13.6	50 180 380	0.4 0.8 1.2	0.4 0.6 1.4	C C C	C C C	ND1	1	Natural soil replicate test
	CL	1734	17.3	50 180 380	0.3 0.6 1.1	0.3 0.7 1.2	C C C	C C C	ND1	1	Natural soil replicate test
	CL	1718	17.3	50 180 380	0.3 0.7 1.2	0.3 0.8 1.2	C C C	C C C	ND1	1	Final mix replicate test
BAF: F-13 Southwest corner	SC	1183	25.5	50 180 380	0.3 0.7 1.2	0.3 0.8 1.2	C C C	C C C	ND1	1	Premix
	SC	1509	25.5	50 180 380	0.3 0.8 1.1	0.3 0.8 1.3	C C C	C C C	ND1	1	Premix replicate test

[a] D = Dark/distinctly cloudy CL = Cloudy SC = Slightly cloudy C = Clear

APPENDIX J
MCGEE CREEK DAM - STOCKPILED LIME-TREATED SOIL

Sample identification	Unified soil class.	Placement d (lb/ft³)	Placement w (%)	Pinhole test data summary Head (mm)	Q_1 (mL/s)	Q_2 (mL/s)	Turbidity[a] Initial	Turbidity[a] Final	Classification	Crumb test	Remarks
BA-F: F-4 Stockpile	SC	1560	17.2	50	0.2	0.4	C	C	ND2	1	2 percent lime-treated soil cured airtight for 22 weeks before test
				180	1.0	1.1	C	C			
				380	1.6	1.7	C	C			
BA-F F-4 Stockpile	SC	1660	16.3	50	0.4	0.4	C	C	ND1	1	Replicate test (see above)
				180	0.8	0.9	C	C			
				380	1.3	1.4	C	C			
BA-F: F-4 Stockpile	SC	1571	9.2	50	0.2	0.2	C	C	ND1	1	2 percent lime-treated soil allowed to air dry for 22 weeks before test
				180	0.5	0.7	C	C			
				380	1.0	1.0	C	C			
BA-F: F-4 Stockpile	SC	1511	9.2	50	0.2	0.2	C	C	ND1	1	Replicate test (see above)
				180	0.7	0.7	C	C			
				380	1.0	1.0	C	C			

APPENDIX K

Procedure for Determining Dispersibility of Clayey Soils by the Pinhole Test Method

USBR 5410-XX

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UNITED STATES DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

USBR 5410. XX *

PROCEDURE FOR

Determining Dispersibility of Clayey Soils
by the Pinhole Test Method

INTRODUCTION

This test procedure is under the jurisdiction of the Geotechnical Branch, code D-1540, Division of Research and Laboratory Services, Engineering and Research Center, Denver, Colorado. The test procedure is issued under the fixed designation USBR 5410. The number immediately following the designation indicates the year of acceptance or the year of last revision.

This test procedure is similar to that outlined in Bureau of Reclamation Report REC-ERC-79-10, pp. 1-8, except for a rewritten, more detailed description of the test procedure, new equipment design features, and new data forms for recording and interpreting results.

1. Scope

1.1 This designation outlines the procedure for determining dispersibility of clayey soils by the pinhole test method.

1.2 Dispersibility, dispersivity, or dispersion refers to the process of clay colloid deflocculation in the presence of water. Deflocculation of clay colloids occurs when the interparticle repulsive forces exceed the attractive forces. These physicochemical forces are very large in relation to the gravity forces on the individual clay particles and, hence, cause the individual particles to go into suspension in the presence of water.

1.3 The pinhole test is a direct, physical method for evaluating the dispersibility of clay soils while water flows through a small pinhole in the soil specimen.

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* This test designation is annotated - XX to indicate preliminary issue of yet to be approved test procedure.

1.4 The dispersion of clay colloids is observed by cloudiness in the discharge effluent.

1.5 For dispersive soils, the measured flow rate increases as the pinhole diameter enlarges until the hydraulic capacity of the equipment has been reached. For nondispersive soils, the measured flow rate remains constant at each head and the pinhole diameter stays the same throughout the test. A detailed analysis of pinhole equipment hydraulic characteristics for dispersive and nondispersive is presented in reference [1].**

1.6 The test should be conducted only on soils with more than 12 percent minus 0.005-mm particles and with plasticity index greater than or equal to 7, as determined in accordance with test designations USBR 5330, 5350, and 5360.

2. Auxiliary Tests

2.1 USBR Test Designations

2.1.1 USBR 1009, Calibrating Proctor Compaction Molds
2.1.2 USBR 5300, Determining Water Content of Soils by the Oven Method
2.1.3 USBR 5330, Performing Particle Size Analysis of Soils, including Hydrometer Analyses
2.1.4 USBR 5350, Determining Liquid Limit of Soils by the One Point Method
2.1.5 USBR 5360, Determining Plastic Limit and Plasticity Index of Soils
2.1.6 USBR 5400, Determining Dispersibility of Clayey Soils by the Crumb Test Method
2.1.7 USBR 5405, Determining Dispersibility of Clayey Soils by the Double Hydrometer Test Method
2.1.8 USBR 5500, Performing Laboratory Compaction of Soils (Moisture Density Relations)

3. Applicable Documents

3.1 USBR 1009, Calibrating Proctor Compaction Molds
3.2 USBR 1012, Calibrating Balances and Scales
3.3 USBR 1025, Calibrating Sieves and Screens
3.4 USBR 5300, Determining Water Content of Soils by the Oven Method
3.5 USBR 5330, Performing Particle Size Analyses of Soils, including Hydrometer Analyses
3.6 USBR 5350, Determining Liquid Limit of Soils by the One Point Method
3.7 USBR 5360, Determining Plastic Limit and Plasticity Index of Soils
3.8 USBR 5400, Determining Dispersibility of Clayey Soils by the Crumb Test Method
3.9 USBR 5405, Determining Dispersibility of Clayey Soils by the Double Hydrometer Test Method

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** Numbers in brackets indicate references.

3.10 USBR 5500, Performing Laboratory Compaction of Soils (Moisture Density Relations)
3.11 USBR 5510, Performing Laboratory Compaction of Soils (Harvard Miniature)
3.12 USBR 9100, List of Laboratory Equipment

4. Summary of Method

4.1 A 25-mm-long by 35-mm-diameter cylindrical soil specimen is secured between two end plates and connected to a constant head tank. The intake end plate incorporates a 1.5-mm-diameter hole that serves as a centering guide for punching a 1.0-mm-diameter hole through the 25-mm-long soil specimen. The 1.0-mm-diameter hole, the "pinhole" for which the test is named, is made with a 6-inch aircraft extension drill. Water is percolated through this pinhole under heads of 50, 180, and 380 mm, and the flow rate and effluent turbidity are recorded. For nondispersive soils, the pinhole diameter remains unchanged throughout the test, the discharge effluent remains clear, and the measured flow rate remains constant at each head. For dispersive soils, the pinhole diameter increases throughout the test, the discharge effluent contains easily visible clay colloids in suspension, and the flow rate at the 50-mm head increases rapidly during the first 10 minutes of the test.

5. Significance and Use

5.1 The pinhole test has become the generally accepted reference test for identifying dispersive clays.

5.2 Water flow through the pinhole in the soil specimen simulates the water flow through a crack in the impervious core of a dam or other structure. The 50-, 180-, and 380-mm heads cause flow velocities ranging from approximately 30 to 160 cm/s at hydraulic gradients ranging from approximately 2 to 15.

5.3 The pinhole test was originally developed, in part, by using clayey soils selected from field sites with known erodible and erosion resistant characteristics. The results of tests conducted on hundreds of specimens indicate that the pinhole test gives the best correlation with erosional performances of soils in nature. Other indirect tests such as the crumb test (USBR 5400), double hydrometer (USBR 5405), and soil-pore water cation and zeta potential analyses are also used as dispersive clay identification tests. However, the results of those tests are less effective than the pinhole test for identifying dispersive clays.

5.4 The pinhole test has been successfully used on the McGee Creek Dam and Central Arizona Projects to identify dispersive and nondispersive soils for selective use in constructing project features.

5.5 The pinhole test is generally used as the control test in research programs for developing new or improved techniques for identifying dispersive soils.

5.6 The pinhole test, as originally developed by Sherard [2], was intended as a qualitative test which emphasized visually observing the discharge water. As a result of USBR research efforts and operating experience, a quantitative evaluation method was developed and documented [1]. The quantitative method emphasizes the measured flow rate as the primary test evaluation criterion. This test procedure is adapted from references [1 and 2].

6. Terminology

6.1 Dispersive Clay - clay colloids that deflocculate in the presence of water because the physicochemical repulsive forces on the particles exceed the attractive forces. Dispersive clay colloids are easily detached from one another, and the individual particles go into suspension in the presence of water.

6.2 Turbidity - visual cloudiness caused by suspended clay colloids in the discharge water collected during the pinhole test.

6.3 Cations - positively charged ions, usually Na, Ca, Mg, and K are the major cations available in a soil-water system.

7. Interferences

7.1 The presence of significant amounts of fine and medium size sand particles in the pinhole test specimen can affect the test results. The larger sand particles may not completely pass through the pinhole specimen after separating from the surface of the pinhole. When this occurs, the flow rate may be reduced and some excessive local scour erosion may occur along the pinhole at the location where the sand particles were separated and/or deposited. Later in the test, the water velocity at the 180- or 380-mm head may be sufficient to move those sand particles and clear up blockages that may have occurred. This phenomenon can only be determined at the conclusion of the test by inspecting the final shape of the eroded pinhole and observing fine or medium sand accumulated at the downstream end of the specimen or in the discharge end plate.

8. Apparatus

8.1 USBR 9100, List of Laboratory Equipment

8.1.1 Balance, Item 040.

8.2.2 Sieve, U.S. Standard Series Nos. 4 and 10 Item 640.

8.2 Pinhole specimen compaction and test apparatus (fig. 6K-1) consisting of:

8.2.1 Modified Harvard miniature compaction mold stand, collar, pinhole specimen mold and specimen mold adapter.

8.2.2 Intake and Discharge End Plates with wing nut and bolt assembly.

8.3 Miscellaneous Items.

8.3.1 Harvard Miniature Compaction Tamper with 20-lb (6.8-kg) spring.

8.3.2 Dowel, wood or plastic, used for leveling loose soil lifts in the pinhole specimen mold.

9. Reagents and Materials

9.1 Use distilled water for preparing the compacted soil specimen and for percolating through the hole in the test specimen.

9.2 The use of tap water or water from other sources is not recommended. It has been shown that distilled water has the greatest potential for exchanging absorbed cations from the clay mineral surfaces [2]. Water with impurities, i.e., water containing cations, anions, and ion pairs in solution, will generally cause a reduced degree of dispersion. The use of distilled water represents the worst case situation for laboratory evaluation of soil dispersibility.

10. Precautions

10.1 Safety Precaution

10.1.1 There are no special safety precautions required.

10.2 Technical Precautions

10.2.1 The test specimen should be prepared and compacted as quickly as possible to avoid unrecorded moisture loss that would reduce placement dry density and moisture content.

11. Sampling, Test Specimens, and Test Units

11.1 Obtain approximately 400 grams of representative minus No. 4 soil in accordance with USBR 5205.

Note 1: 400 grams of soil are required to provide sufficient material for concurrently performing the crumb and double hydrometer tests, USBR 5400 and USBR 5405, respectively, in addition to the pinhole. USBR 5400, 5405, and 5410 generally constitute the dispersive clay test series.

11.2 Preserve the soil specimen at natural water content, if possible, or allow to air dry only. Oven-dried soil should not be used because of the possible effects on changing the dispersion potential of some clay minerals.

11.3 Obtain approximately 150 grams of minus No. 10 soil.

12. Calibration and Standardization

12.1 Check to see that the pinhole specimen mold is currently calibrated in accordance with USBR 1009, Calibrating Proctor Compaction Mold Volumes.

If the calibration is not current, the calibration procedures should be performed before using the pinhole specimen mold.

12.2 Check to see that the balance to be used for weighing the soil specimen is calibrated in accordance with USBR 1012, Calibrating Balances and Scales. If the calibration is not current, the calibration procedures shuld be performed before using the balance.

12.3 Check to see that the sieves to be used for preparing the soil specimen are calibrated in accordance with USBR 1025, Calibrating Sieves and Screens. If the calibration is not current, the calibration procedures should be performed before using the sieves.

13. Conditioning

13.1 Determine the Proctor maximum dry density and optimum water content of the soil by performing USBR 5500, Performing Laboratory Compaction of Soils, or by consulting the engineer or supervisor in charge.

13.2 Determine 98 percent Proctor maximum dry density and the moisture content 2 percent dry of optimum water content.

Note 2: Pinhole test specimens are generally compacted at 98 percent Proctor maximum dry density and 2 percent dry of optimum water content. Variations should be approved by the engineer or supervisor in charge.

13.3 Determine the water content of a representative portion of the 150-gram soil sample by performing USBR 5300, Determining Water Content of Soils by the Oven Method.

13.4 Calculate the amount of water to be added to the remaining soil sample to increase the water content to the required value.

13.5 Allow the moist soil sample to mellow for a minimum of 16 hours to ensure uniform distribution of water throughout the sample.

13.6 Determine the water content of a representative portion of the 150-gram soil sample by performing USBR 5300, Determining Water Content of Soils by the Oven Method.

13.7 Repeat paragraphs 13.4 through 13.6 until the measured water content is within ± 1 percent of the desired water content.

14. Procedure

14.1 Select the pinhole specimen mold to be used for compacting the test specimen. Record its calibrated volume.

14.2 Determine the weight of moist soil that is required to achieve the desired placement density in the selected pinhole specimen mold using figure X1.1.

14.3 Place the container that will hold the moist soil on the scale, and record its weight or tare the scale to zero.

14.4 Add the total amount of moist soil required to the container.

14.5 Divide the total amount of moist soil by 5, the required number of lifts. This is the amount of soil required for each lift.

14.6 Remove the amount of moist soil required for the first lift and place it into the assembled pinhole specimen mold, mold collar, and compaction stand (fig. 6K-1) and gently level the surface using the wood or plastic dowel. Record the total weight of moist soil remaining in the container.

14.7 Compact the soil using 25 tamps of the Harvard Miniature Spring-Loaded Tamper. Distribute the tamper to ensure 100 percent coverage of the soil specimen.

14.8 Remove the amount of moist soil required for a second lift, place it into the mold and gently level the surface with the dowel. Record the total weight of moist soil remaining in the container.

14.9 Repeat paragraphs 14.6 through 14.8 until the container has been completely emptied.

14.10 After the final lift is compacted, carefully remove the pinhole specimen mold from the collar and mold assembly.

Note 3: An alternate compaction procedure, adapted from reference [3], may be used. It eliminates weighing the soil required for each lift. For this procedure, use the 2 slightly heaping teaspoons of soil per lift, with 3 total lifts required and 25 tamps per lift. The proper amount of soil for each lift may vary with different soils. Additional trials may be required to develop the technique for selecting the proper amount of soil.

14.11 Inspect the top of the specimen (the most recently compacted surface) to ensure that the soil completely fills the mold with little or no excess soil.

14.12 Carefuly trim excess soil from the top of the specimen using a straightedge or fine-tooth hacksaw blade and brush all soil from the mold surfaces.

14.13 Weigh the specimen and mold assembly and determine the dry density. If the actual placement dry density is greater than or equal to the desired density (usually 98 percent Proctor maximum), retain the test specimen. If the actual placement density is too low, the specimen must be rejected and another specimen prepared in accordance with paragraphs 14.2 through 14.2, using an additional five tamps per lift.

14.14 Record the placement density obtained, the number of tamps per lift, and the number of lifts required for final specimen compaction using figure 6-K2.

14.15 Place the top of the specimen and mold in the intake end plate. Place a No. 10 screen on the bottom of the specimen and press on the discharge end plate. Close the assembly using the bolts and wing nuts. Alternately and lightly tighten each wing nut making the end plates close evenly. The watertight seal is maintained by the O-ring seals in the end plates, not the torque on the wing nuts.

14.16 Carefully punch the pinhole into the specimen, from the intake end plate completely through to the discharge end plate, using a No. 40 6-inch drill and pliers. When the drill appears through the discharge end plate, pull it completely through, using the pliers in one slow continuous motion to ensure that a uniform, 1.0-mm-diameter pinhole is obtained.

Note 4: Occasionally, the path of the drill is changed by sand particles so that it hits the screen or face of the discharge end plate. When this occurs, do not force the drill. Instead, remove the discharge end plate and determine if the drill hit the screen or the discharge end plate. If the drill hit the screen, move the screen slightly, reattach the discharge end plate and continue punching the pinhole. If the drill hit the end plate, carefully pull the drill through the specimen and the screen with the pliers and reattach the discharge end plate.

14.17 After the pinhole is punched, remove the screen from the bottom of the specimen and reassemble the discharge end plate to the intake end plate and specimen.

14.18 Close the outlet valve of the constant head tank, fill with distilled water, and secure rubber stopper in the top of the tank. Connect the constant head tank outlet to the intake end plate. Connect the standpipe tube to the vent in the intake end plate. Secure the assembled end plates and mold in the horizontal position.

14.19 Open the bleed valve in the head tank outlet line until the water level in the air intake tube lowers to the bottom of the tube and air enters the head tank through the air intake tube.

14.20 Start the test by slowly opening the head tank outlet valve and raising the position of the head tank until the head water reaches the 50-mm mark on the standpipe tube.

14.21 Record the time at the start of the test using figure 6K-3.

14.22 Measure the flow rate in mL/s using a 100-mL graduate and a stopwatch. Collect the effluent for 60-second intervals, record the amount of milliliters collected, and divide by 60 to determine mL/s.

14.23 Observe the cloudiness (turbidity) of the effluent and record for each 60-second interval as either dark, cloudy, slightly cloudy, or clear. Observe the effluent through the side of the graduate and from the top of the graduate. Clear effluent can only be judged from the top view.

NOTE 5: If flow does not begin at the start of the test or if flow stops during the test, stop the test, remove the assembled apparatus, repunch the pinhole, and continue the test.

14.24 Continue the test for 5 minutes at the 50-mm head. If the flow is dark or cloudy, and flow rates have gradually increased to 1.0-1.4 mL/s, the test is complete.

14.24.1 Dismantle the apparatus and determine the final size of the eroded pinhole by either extruding the specimen and splitting it longitudinally or observing the pinhole from each end. The final hole diameter will generally be two or more times the original 1.0-mm-diameter.

14.24.2 Classify the soil as dispersive, D1.

14.25 If the flow at the 50-mm head is dark, cloudy, slightly cloudy, or clear at the end of 5 minutes and the flow rate is less than 1.0 mL/s, continue the test for an additional 5 minutes, at the 50 mm head.

14.25.1 If the flow at the 50-mm head is dark, cloudy, or slightly cloudy and the flow rate has increased to 0.80-1.4 mL/s, the test is complete.

14.25.2 Classify the soil as dispersive - D2 - if the flow rate is 1.0-1.4 mL/s. Classify the soil as intermediate - ND4 - if the flow rate is 0.80-1.0 mL/s.

14.26 If the flow at the 50-mm head is clear or slightly cloudy and the flow rate is 0.20-0.80 mL/s, raise the head to 180 mm and continue the test for an additional 5 minutes.

14.26.1 If the flow at the 180-mm head is cloudy or slightly cloudy and the flow rate has increased to 1.2-2.8 mL/s, the test is complete.

14.26.2 Dismantle the apparatus and determine the final size of the eroded pinhole by either extruding the specimen and splitting it longitudinally or observing the pinhole from each end. The final hole diameter will generally be 1.5 to 2 times the original 1.0-mm diameter.

14.26.3 Classify the soil as intermediate - ND4 - if the flow rate is 2.0-2.8 mL/s. Classify the soil as intermediate - ND3 - if the flow rate is 1.2-2.0 mL/s.

14.27 If the flow at the 180-mm head is clear or slightly cloudy and the flow rate is 0.6-1.2 mL/s, raise the head to 380 mm and continue the test for an additional 5 minutes.

14.28 If the flow at the 380-mm head is clear or slightly cloudy and the flow rate is 0.80-2.5 mL/s, the test is complete.

14.28.1 Dismantle the apparatus and determine the final size of the pinhole by either extruding the specimen and splitting it longitudinally or observing the pinhole from each end. The final hole diameter will generally be 1 to 1.5 times the original 1.0-mm diameter.

14.28.2 Classify the soil as nondispersive - ND2 - if the flow rate is 1.6-2.5 mL/s. Classify the soil as nondispersive - ND1 - if the flow rate is 0.80-1.6 mL/s.

15. Interpretation of Results

15.1 For each 60 second flow measurement interval, record the water turbidity, the volume of water collected (in mL), calculate the flow rate in mL/s, and plot the flow rate on figure 6K-4.

16. Report

16.1 The pinhole test report includes the following:

16.1.1 Project/feature name.

16.1.2 Soil samples index number or depth and location of sample.

16.1.3 Water content of sample before sample preparation.

16.1.4 Water content and placement dry density of compacted specimen including number of lifts and number of tamps per lift.

16.1.5 Hydraulic head in millimeters, turbidity of flow at each measurement interval, and flow rate in mL/s at each measurement interval.

16.1.6 Description, sketch, or photograph of final pinhole at the completion of the test.

16.1.7 Final dispersive grade classifications, D1, D2, ND4, ND3, ND2, and ND1.

16.2 Example and blank data sheets for dispersive and nondispersive pinhole test results are given on figures 6K-2 through 6K-10.

16.3 Completing figures 6K-2, 6K-3, and 6K-4 satisfies all the requirements of the pinhole test report.

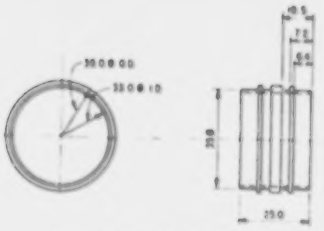
REFERENCES

[1] Acciardi, Raymond G., "Quantification of Pinhole Test Equipment Hydraulic Characteristics," Technical Report REC-ERC-82-15, Bureau of Reclamation, Denver, Colorado, September 1982.

[2] Sherard, James L. et al., "Pinhole Test for Identifying Dispersive Soils," Journal of the Geotechnical Division, ASCE, vol. 102, No. GT1, Proc. Paper No. 11846, pp. 69-85, January 1976.

[3] Soiltest Inc., Operating Instructions for Harvard Miniature Compaction Apparatus, Model No. CN-435, May 1977.

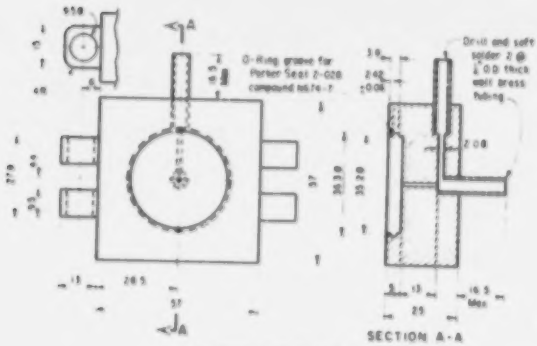
USBR 1009, Calibrating Proctor Compaction Molds
USBR 5300, Determining Water Content of Soils by the Oven Method
USBR 5330, Performing Particle Size Analyses of Soils, including Hydrometer Analyses
USBR 5350, Determining Liquid Limit of Soils by the One Point Method
USBR 5360, Determining Plastic Limit and Plasticity Index of Soils
USBR 5400, Determining Dispersibility of Clayey Soils by the Crumb Test Method
USBR 5405, Determining Dispersibility of Clayey Soils by the Double Hydrometer Test Method
USBR 5500, Performing Laboratory Compaction of Soils (Moisture Density Relations)
USBR 5510, Performing Laboratory Compaction of Soils (Harvard Miniatures)
USBR 9100, List of Laboratory Equipment



PINHOLE SPECIMEN MOLD

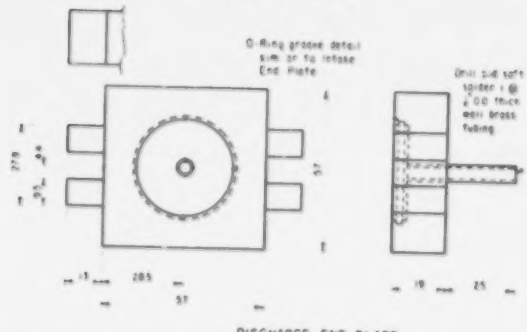
SPECIMEN MOLD ADAPTER

NOTES
1. All dimensions are in millimeters.
2. Ø indicates diameter.
3. Stainless steel stock required.
4. Specimen Mold Adaptor is designed for use with Soiltest, Inc. Part No CN 428, Mold Holder. The adaptor allows the Standard Hazard Miniature Compaction Mold Holder (CN 428) to be used with the U.S.B.R. Pinhole Specimen Mold.



INTAKE END PLATE

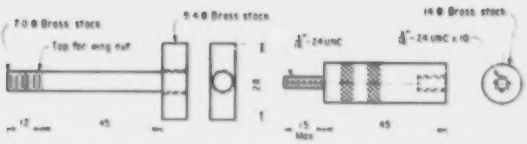
SECTION A-A



DISCHARGE END PLATE

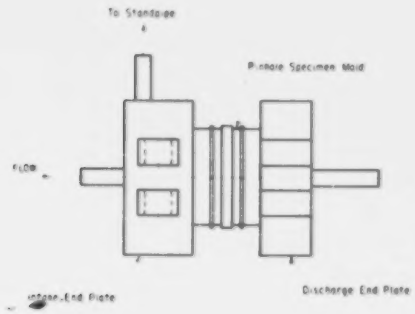
NOTES
1. All dimensions are in millimeters.
2. Ø indicates diameter.
3. Brass stock required.

Figure 6K-1. - USBR pinhole test apparatus (sheet 1 of 2).

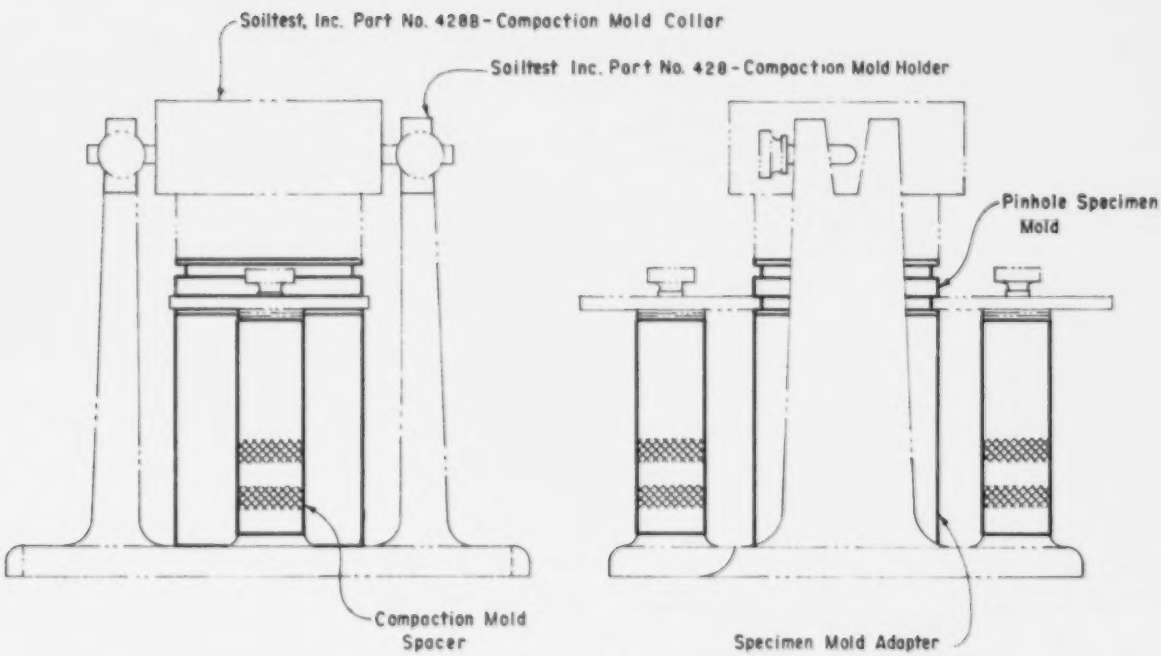


WING NUT & BOLT ASSEMBLY
(END PLATE CLAMPING DEVICE)

COMPACTION MOLD SPACER



PINHOLE TEST APPARATUS - ASSEMBLY DRAWING
(WING NUT & BOLT ASSEMBLY OMITTED FOR CLARITY)


Soiltest, Inc. Part No. 428B - Compaction Mold Collar
Soiltest Inc. Part No. 428 - Compaction Mold Holder
Pinhole Specimen Mold
Compaction Mold Spacer
Specimen Mold Adapter

PINHOLE SPECIMEN COMPACTION APPARATUS - ASSEMBLY DRAWING
(COMMERCIALLY AVAILABLE ITEMS SHOWN IN PHANTOM LINES)

Figure 6K-1. - USBR pinhole test apparatus (sheet 2 of 2).

PINHOLE TEST

PROJECT _____ TESTED BY _____ DATE _____
FEATURE _____ COMPUTED BY _____ DATE _____
SAMPLE NO. _____ CHECKED BY _____ DATE _____

MOLD NO. _____ VOLUME OF MOLD _____ cm^3 WEIGHT OF MOLD _____ gms

---

COMPACTION DATA FOR REMOLDED SPECIMENS

1.	Dry Density (pcf ÷ 0.01602) = g/cm^3	_____ g/cm^3
2.	Water Content	_____ g
3.	Initial Dry Wt.	_____ g
4.	Initial Wet Wt.	_____ g
5.	Wet Wt. ÷ 5 layers	_____ g
6.	Wet Wt. + Container _____ g	_____ g
7.	Total Wt. Less Each Layer	(1) _____ g
		(2) _____ g
		(3) _____ g
		(4) _____ g
		(5) _____ g

ACTUAL OBTAINED DENSITY OF REMOLDED OR UNDISTURBED SPECIMEN

8.	Wet Wt. + Mold	_____ g
9.	Wt. of Mold	_____ g
10.	Wt. of Wet Soil	_____ g
11.	Wet Density	_____ g/cm^3
12.	Dry Density	_____ g/cm^3
13.	Blows Per Layer	_____

NOTES:

EROSION DATA

CROSS-SECTION OF SPECIMEN

14.	Dish #	_____
15.	Dish Wt.	_____ g
16.	Dry Wt. + Mold + Dish	_____ g
17.	Wt. of Mold + Dish	_____ g
18.	Wt. of Dry Soil	_____ g
19.	$\left[\frac{\text{Actual Dry Wt. - Final Dry Wt}}{\text{Actual Dry Wt.}}\right] \times 100$	_____ %

U/S _____ mm D/S _____ mm

MOISTURE DATA

Dish No.	_____
Wet Soil + Dish	_____ g
Dry Soil + Dish	_____ g
Wt. of Dish	_____ g
Wt. of Water	_____ g
Wt. of Dry Soil	_____ g
Moisture	_____ %

NOTES

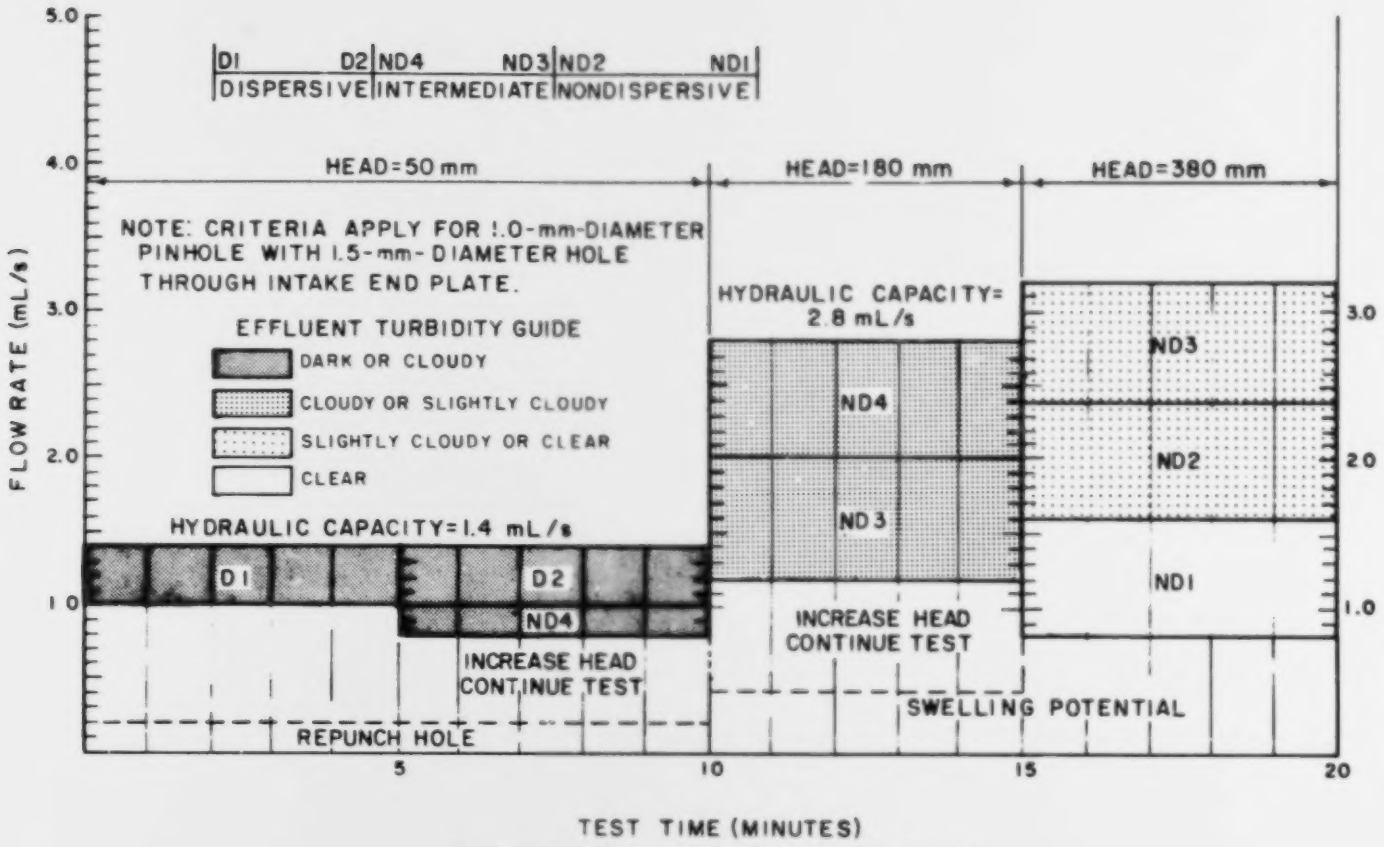
Figure 6K-2. - Pinhole specimen compaction and erosion data.

Sample _____ Sheet No. 2

BOTTLE NO.	TIME	HEAD ** MM	FLOW RATE			TURBIDITY NTU	CONDUCTANCE μSIEMENS/cm K= @ TEMP °C	PH	NA+ PPM	WATER QUALITY*
			ML	SEC	ML/SEC					

* WATER QUALITY SYMBOLS: C=CLEAR SC=SLIGHTLY CLOUDY CL=CLOUDY D=DARK **HEAD TANK TEMPERATURE:

Figure 6K-3. - Pinhole test flow rate data.



DISPERSIVE GRADE VS FLOW RATE

SAMPLE NUMBER	PLACEMENT CONDITIONS		DISPERSIVE GRADE CLASSIFICATION
	DRY DENSITY	WATER CONTENT	

Figure 6K-4. - Pinhole test dispersive grade versus flow rate.

PINHOLE TEST

PROJECT CENTRAL ARIZONA PROJECT
FEATURE TUCSON AQUEDUCT - REACH 1
SAMPLE NO. AP-137-TA1 20.0'-25.0'

TESTED BY RPB DATE 8/11/83
COMPUTED BY AP DATE 8/11/83
CHECKED BY RPB DATE 8/11/83

MOLD NO. B VOLUME OF MOLD 21.64 cm^3 WEIGHT OF MOLD 27.8 gms

COMPACTION DATA FOR REMOLDED SPECIMENS

1.	Dry Density (pcf ÷ 0.01602) = g/cm^3	1.854 g/cm^3
2.	Water Content	10.8 %
3.	Initial Dry Wt.	- g
4.	Initial Wet Wt.	40.10 g
5.	Wet Wt. ÷ 5 layers	8.02 g
6.	Wet Wt. + Container 90.1 g (50.0 g cont)	90.1 g
7.	Total Wt. Less Each Layer	
	(1)	82.08 g
	(2)	74.06 g
	(3)	66.04 g
	(4)	58.02 g
	(5)	50.00 g

ACTUAL OBTAINED DENSITY OF REMOLDED OR UNDISTURBED SPECIMEN

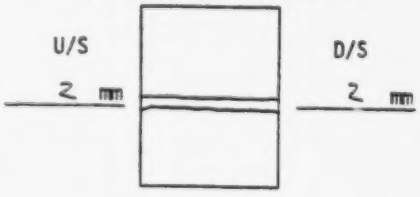
8.	Wet Wt. + Mold	73.10 g
9.	Wt. of Mold	27.8 g
10.	Wt. of Wet Soil	45.3 g
11.	Wet Density	2.094 g/cm^3
12.	Dry Density	1.890 g/cm^3
13.	Blows Per Layer	25

NOTES: PROCTOR MAX DRY DENSITY IS 1951 gm/cm^3
OPTIMUM WATER CONTENT IS 12.3 %

EROSION DATA

14.	Dish #	
15.	Dish Wt.	g
16.	Dry Wt. + Mold + Dish	g
17.	Wt. of Mold + Dish	g
18.	Wt. of Dry Soil	g
19.	$\left[\frac{\text{Actual Dry Wt. - Final Dry Wt.}}{\text{Actual Dry Wt.}}\right] \times 100$	%

CROSS-SECTION OF SPECIMEN



MOISTURE DATA

Dish No.	201
Wet Soil + Dish	249.8 g
Dry Soil + Dish	247.2 g
Wt. of Dish	223.0 g
Wt. of Water	2.6 g
Wt. of Dry Soil	24.2 g
Moisture	10.8 %

NOTES
EROSION DATA NOT OBTAINED

Figure 6K-5. - Pinhole specimen compaction and erosion data (typical dispersive soil).

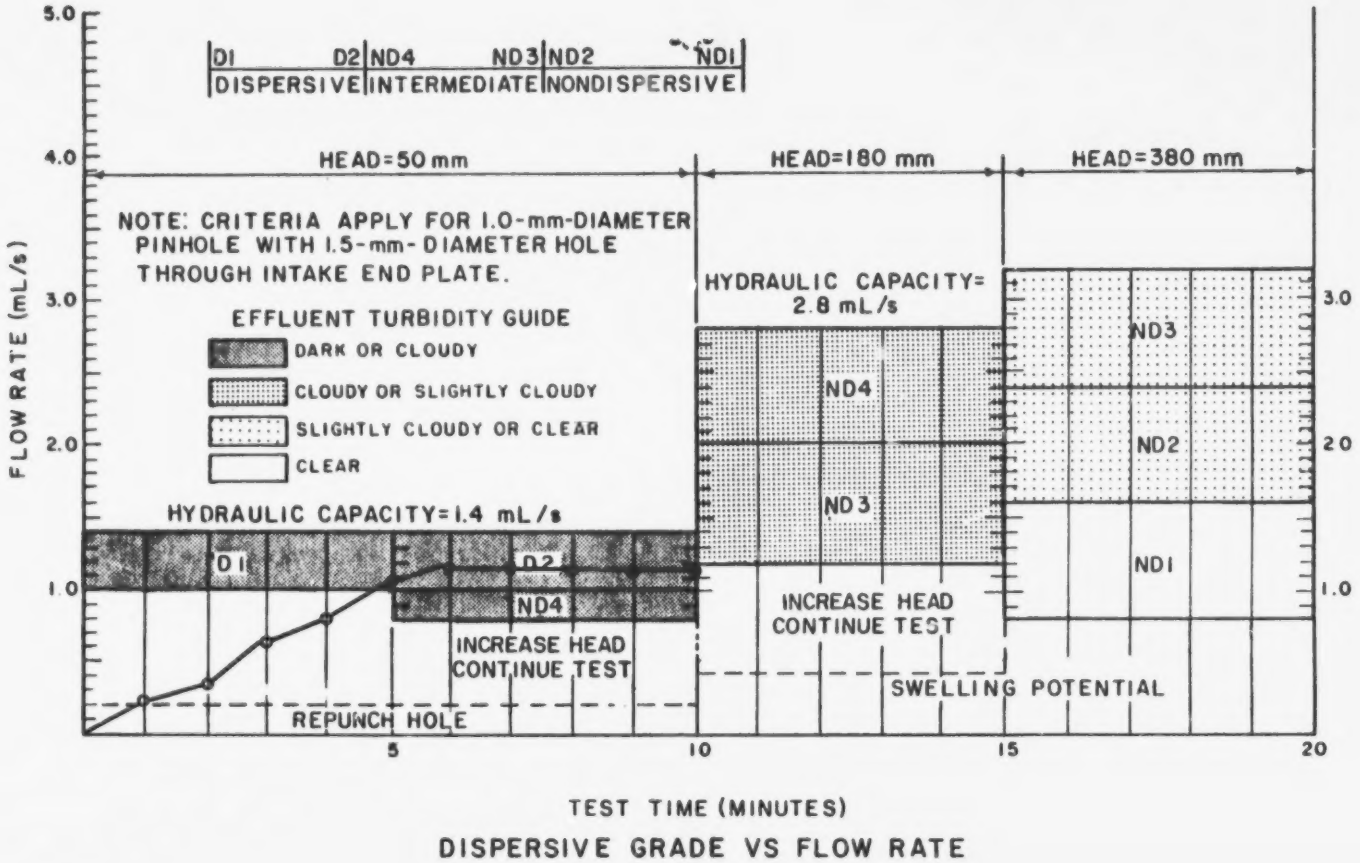
Sample AP-137-TA1 20.0'-25.0' 8/11/83 Sheet No. 2

BOTTLE NO.	TIME	HEAD ** MM	FLOW RATE ML	SEC	ML/SEC	TURBIDITY NTU	CONDUCTANCE µSIEMENS/cm K= @ TEMP °C	PH	NA+ PPM	WATER QUALITY*
	1007	50	14	60	0.23	SC				
			20	60	0.33	SC				
			37	60	0.62	SC				
			49	60	0.82	SC				
			61	60	1.02	SC				
			69	60	1.15	SC				
			69	60	1.15	SC				
			69	60	1.15	SC				
			68	60	1.13	SC				
			70	60	1.17	SC				

* WATER QUALITY SYMBOLS: C=CLEAR SC=SLIGHTLY CLOUDY CL=CLOUDY D=DARK

**HEAD TANK TEMPERATURE:

Figure 6K-6. - Pinhole test flow rate data (typicial dispersive soil).



DISPERSIVE GRADE VS FLOW RATE

SAMPLE NUMBER	PLACEMENT CONDITIONS		DISPERSIVE GRADE CLASSIFICATION
	DRY DENSITY	WATER CONTENT	
AP-137-TA1	1.890 gm/cm³	10.8%	D1

Figure 6K-7. - Pinhole test dispersive grade versus flow rate (typical dispersive soil).

PINHOLE TEST

PROJECT McGee Creek
FEATURE McGee Creek
SAMPLE NO. 56C-199

TESTED BY Kinkaid
COMPUTED BY Kinkaid
CHECKED BY Kienitz

COMPACTION DATA

MOLD NO. 3 VOLUME OF MOLD 21.58 cm^3 WEIGHT OF MOLD 28.54 gms

1. Dry Density	1.79	g/cm^3
2. Water Content	14.8	%
3. Initial Dry Wt.	38.63	g
4. Initial Wet Wt.	44.35	g
5. Wet Wt. ÷ 5 Layers	8.87	g
6. Wet Wt. + Container 27.45g	71.80	g
7. Total Wt. Less Each Layer	(1) 62.93	g
	(2) 54.06	g
	(3) 45.19	g
	(4) 36.32	g
	(5) 27.45	g

Actual Density After Compaction

8. Wet Wt. + Mold	73.23	g
9. Wt. of Mold	28.54	g
10. Wt. of Wet Soil	44.69	g
11. Wt. of Dry Soil	38.93	g
12. Dry Density	1.80	g/cm^3
13. Blows Per Layer	20	

NOTES:

EROSION DATA

14. Dry Wt. + Mold	67.10	g
15. Wt. of Mold	28.54	g
16. Wt. of Dry Soil	38.56	g
17. [Initial Dry Wt. - Final Dry Wt/Initial Dry Wt.]	.2	%
18. Hole Size	1	mm

MOISTURE DATA

DISH NO.	88	
Wet Soil + Dish	179.7	g
Dry Soil + Dish	177.9	g
Wt. of Dish	139.1	g
Wt. of Water	1.8	g
Wt. of Dry Soil	38.8	g
Moisture	4.6	%

First added to dry soil to M.C. [illegible]

NOTES:

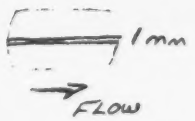


Figure 6K-8. - Pinhole specimen compaction and erosion data (typical nondispersive soil).

Sample 56C-119 Sheet No. 2

BOTTLE NO.	TIME	HEAD ** MM	FLOW RATE ML	SEC	ML/SEC	TURBIDITY NTU	CONDUCTANCE MICROMHOS/cm K=	@ TEMP °C	PH	NA+ PPM	WATER QUALITY*
		50	21	60	0.35						C
			21	60	0.35						C
			21	60	0.35						C
			21	60	0.35						C
			21	60	0.35						C
			21	60	0.35						C
			21	60	0.35						C
			21	60	0.35						C
			21	60	0.35						C
			21	60	0.35						C
			21	60	0.35						C
			21	60	0.35						C
		180	47	60	0.78						C
			50	60	0.83						C
			50	60	0.83						C
			51	60	0.85						C
			51	60	0.85						C

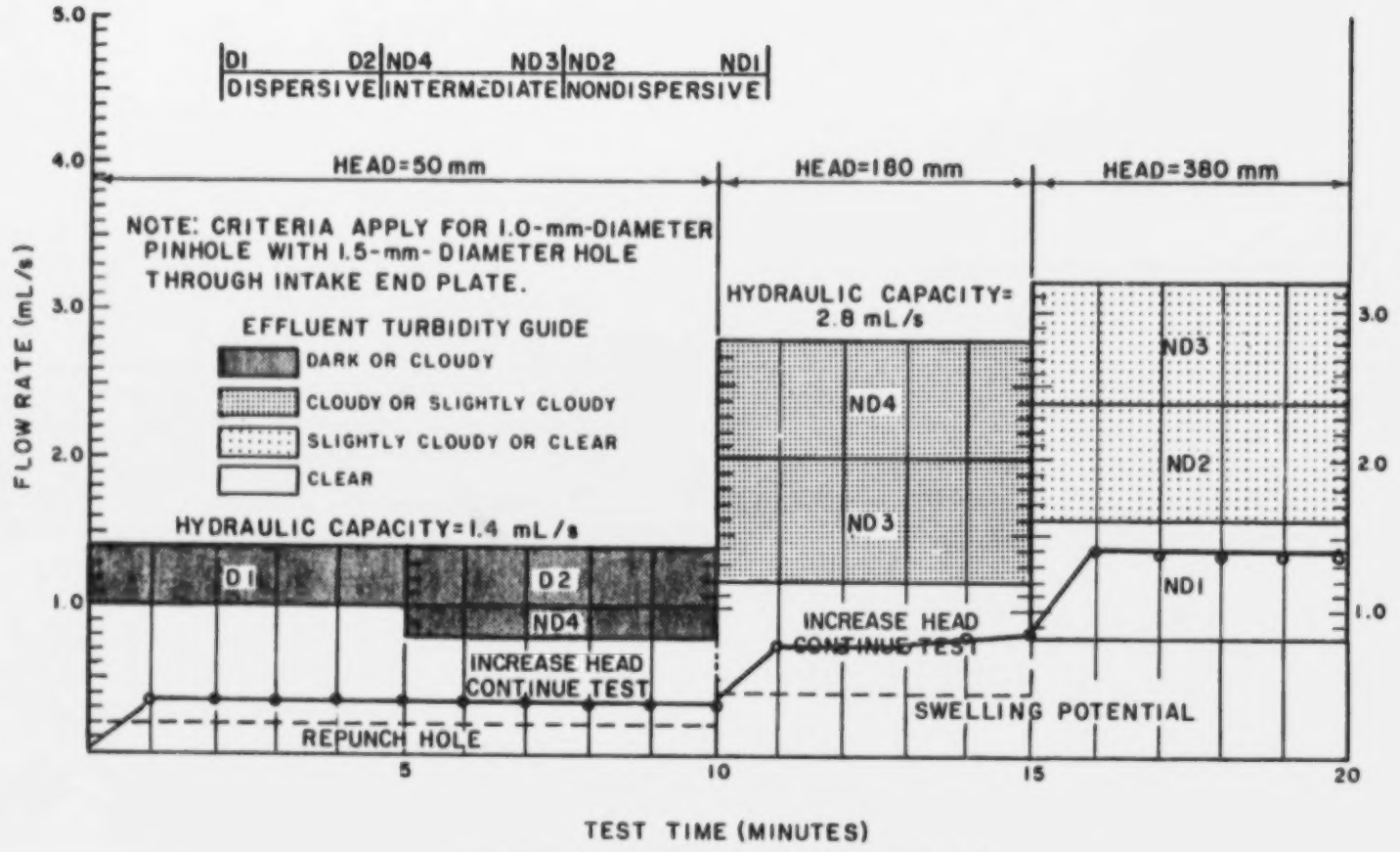
* WATER QUALITY SYMBOLS: C=CLEAR SC=SLIGHTLY CLOUDY CL=CLOUDY D=DARK
**HEAD TANK TEMPERATURE:

Sample 56C-117 Sheet No. 2

BOTTLE NO.	TIME	HEAD ** MM	FLOW RATE ML	SEC	ML/SEC	TURBIDITY NTU	CONDUCTANCE MICROMHOS/cm K=	@ TEMP °C	PH	NA+ PPM	WATER QUALITY*
		340	86	60	1.4						C
			82	60	1.4						C
			82	60	1.4						C
			82	60	1.4						C
			82	60	1.4						C

* WATER QUALITY SYMBOLS: C=CLEAR SC=SLIGHTLY CLOUDY CL=CLOUDY D=DARK
**HEAD TANK TEMPERATURE:

Figure 6K-9. - Pinhole test flow rate data (typical nondispersive soil).



DISPERSIVE GRADE VS FLOW RATE

SAMPLE NUMBER	PLACEMENT CONDITIONS		DISPERSIVE GRADE CLASSIFICATION
	DRY DENSITY	WATER CONTENT	
56C-199	1.80 g/cm³	14.8 %	ND1

Figure 6K-10. - Pinhole test dispersive grade versus flow rate (typical nondispersive soil).

APPENDIX L

Procedure for Determining the Dispersibility of Clayey Soils by the Crumb Test Method

USBR 5400-XX

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UNITED STATES DEPARTMENT OF THE INTERIOR
BUREAU OF RECLAMATION

USBR 5410. XX*

PROCEDURE FOR

Determining Dispersibility of Clayey Soils by the
Crumb Test Method

INTRODUCTION

This procedure is under the jurisdiction of the Geotechnical Branch, code D-1540, Division of Research and Laborabory Services, Engineering and Research Center, Denver, Colorado. The test procedure is issued under the fixed designation USBR 5400. The number immediately following the designation indicates the year of acceptance or the year of last revision.

This procedure is similar tc that outlined in Bureau of Reclamation Report REC-ERC-79-10, except for a rewritten, more detailed description of the procedure photographs of test specimens during the test, and assigned crumb test dispersion grades.

1. Scope

1.1 This designation outlines the procedure for determining the dispersibility of clayey soils by the crumb test method.

1.2 The crumb test has some limitations on its usefulness as an indicator of dispersive clay. A dispersive soil may sometimes give a nondispersive reaction in the crumb test. Soils containing kaolinite with known field dispersion problems have shown nondispersive reactions in the crumb test [1]**. However, if the crumb test indicates dispersion, the soil is probably dispersive.

---

* This test designation is annotated - XX to indicate preliminary issue of yet to be approved test procedure.
** Numbers in brackets indicate references.

1.3 Ovendried soil should not be used to prepare crumb test specimens, as irreversible changes could occur to the soil pore-water physico-chemistry responsible for dispersion [1].

1.4 Generally, the crumb test should be conducted with the pinhole test and the double hydrometer test, USBR 5405 and 5410, respectively. In cases where the results of the pinhole, crumb, and double hydrometer tests disagee, the general order of reliability should be pinhole, followed by crumb and double hydrometer.

2. Applicable Documents

2.1 USBR 5205, Preparing Representative Soil Samples for Laboratory Use

2.2 USBR 5405, Determining Dispersibility of Clayey Soils by the Double Hydrometer Test Method

2.3 USBR 5410, Determining Dispersibility of Clayey Soils by the Pinhole Test Method

2.4 USBR 9100, List of Laboratory Equipment

3. Significance and Use

3.1 The crumb test provides a simple, quick method for field or laboratory identification of dispersive clay.

3.2 The crumb test, as originally developed by Emerson [2], was called the aggregate coherence test and had seven different categories of soil-water reactions. Sherard [3], later simplified the test by combining some soil-water reactions to use only four categories, or grades of soil dispersion.

3.3 The crumb test has successfully been used as an auxiliary test, with the pinhole test as the primary test, on USBR project feature preliminary invesigations of soil dispersibility. As previously noted in this designation, the crumb test should generally be conducted in conjunction with the pinhole and double hydrometer tests.

4. Terminology

4.1 Dispersive clay - clay coloids that deflocculate in the presence of water because the physico-chemical repulsive forces on the particles exceed the attractive forces. Dispersive clay colloids are easily detached from one another, and the individual particles go into suspension in the presence of water.

4.2 Turbidity - visual cloudiness caused by suspended clay colloids in the water.

5. Apparatus

5.1 USBR 9100, List of Laboratory Equipment.

5.1.1 Dish, porcelain, evaporating, 300 mL capacity, item 212.

6. Precautions

6.1 Technical precautions

6.1.1 Carefully place the soil crumb specimen in the water directly on the bottom of the porcelain dish when submerging the crumb. Dropping the crumb from the water surface can cause excessive slaking and erroneous interpretation of the dispersion grade.

6.1.2 Do not jar or move the soil specimen and porcelain dish during the test.

6.1.3 Do not use ovendried soil for crumb test specimens.

6.1.4 The use of natural, irregular V-shaped soil crumbs may result in a specimen that is not representative of the total sample. Therefore, select several crumb test specimens to ensure that representative test results are obtained.

7. Sampling, Test Specimens, and Test Units

7.1 Two methods of obtaining test specimens are available:

7.1.1 A cube, 15 mm on each side, is prepared from moist soil passing the No. 4 sieve. Normally, 400 g of minus No. 4 soil are obtained in accordance with the procedures outlined in USBR 5205, Preparing Representative Soil Samples for Laboratory Use. This 400-g representative sample contains enough soil to conduct crumb, pinhole, and double hydrometer tests. The crumb test specimen can be prepared when the 400-g soil sample has been wetted and cured at the water content desired for compacting the pinhole test specimen. As an alternate method, the crumb test specimen can be prepared as a separate specimen by adding water until wetted enough to prepare a coherent 15-mm cube of soil.

7.1.2 A natural, irregular-shaped soil crumb that has not been allowed to air dry is selected. This soil crumb should be approximately the same volume as a cube 15 mm on each side.

8. Calibration and Standardization

8.1 Not applicable - no calibration or standardization required.

9. Conditioning

9.1 Preserve crumb test specimens at field water content or allow specimens to air dry.

9.2 Do not use ovendried soil for crumb test specimens.

10. Procedure

10.1 Place a 300-mL porcelain evaporating dish near the top portion of the crumb test data sheet with applicable test specimen information noted. See figure 6L-1 for example data sheet and figure 6L-2 for blank data sheet.

10.2 Fill the dish with approximately 250 mL of distilled water.

10.3 Prepare a crumb test specimen by lightly compressing a moist soil sample into a cube 15 mm on each side. Use your thumbs and index fingers to gently form the cube.

NOTE 1. - A natural, irregular-shaped soil specimen may be used in lieu of the specimen described above. For this case, several specimens shold be selected to ensure that test results obtained are representative of the total sample.

10.4 Carefully place the specimen in the water and gently release it only after placing it directly on the bottom of the dish.

10.5 Note and record the time using figure 6L-2.

10.6 As the soil crumb begins to hydrate, observe the tendency for colloidal-sized particles to deflocculate and go into suspension.

NOTE 2. - The colloidal-sized particles are visible in the form of a dense "cloud," or halo, that extends from the soil crumb. The extent and turbidity of the suspended clay particle "cloud" is visually interpreted and is influenced by soil color, i.e., dark soils produce a darker, more distinctly visible cloud.

10.7 Compare the reaction of the soil crumb to the following criteria:

10.7.1 Grade 1. - No reaction - the soil may crumble, slake, diffuse, and spread out, but there is no turbid water created by colloids suspended in the water. All particles settle during the first hour.

10.7.2 Grade 2. - Slight reaction - this is the transition grade. A faint, barely visible colloidal suspension causes turbid water near portions of the soil crumb surface or all around the surface. If the cloud is easily visible, assign grade 3. If the cloud is faintly seen in only one small area, assign grade 1.

10.7.3 Grade 3. - Moderate reaction - an easily visible cloud of suspended clay colloids is seen around all of the outside soil crumb surface. The cloud may extend up to 10 mm away from the soil crumb mass along the bottom of the dish.

10.7.4 Grade 4. - Stong reaction - a dense, profuse cloud of suspended clay colloids is seen around the entire bottom of the dish. Occasionally, the soil crumb dispersion is so extensive that it is difficult to determine the interface of the soil crumb and the colloidal suspension. Often, the colloidal suspension is easily visible on the sides of the dish.

10.8 Refer to figure 6L-3 for photographs of soil crumb specimens and the assigned dispersion grades.

10.9 Determine and record the soil crumb dispersion grade at 2 minutes, 15 minutes, 1 hour, and 24 hours.

11. Calculation and Interpretation of Results

11.1 Use the following criteria to classify crumb test results:

11.1.1 Grade 1. - Nondispersive

11.1.2 Grade 2. - Intermediate

11.1.3 Grades 3 and 4. - Dispersive

11.2 The 1-hour reading is normally used for the overall test evaluation, if the dispersive grade changed during the test.

12. Report

12.1 Complete figure 6L-2 for all crumb test specimens for one sample.

REFERENCES

[1] Holmgren, G.C.S., and C. P. Flanagan, "Factors Affecting Spontaneous Dispersion of Soil Materials as Evidenced by the Crumb Test," Proc., ASTM, Symposium on Dispersive Clays, STP 623, Chicago, Illinois, pp. 218-239, 1976.

[2] Emerson, W. W., "A Classification of Soil Aggregates Based on Their Coherence in Water," Australian Journal of Soil Research, vol. 2, pp. 211-217, 1967.

[3] Sherard, J. L., et al., "Identification and Nature of Dispersive Soils," Journal of the Geotechnical Division, ASCE vol. 102, No. GT4, Proc. Paper 12052, pp. 287-301, April 1976.

USRB 5205, Preparing Representative Soil Samples for Laboratory Use
USBR 5405, Determining Dispersibility of Clayey Soils by the Double Hydrometer Test Method
USBR 5410, Determining Dispersibility of Clayey Soils by the Pinhole Test Method
USBR 9100, List of Laboratory Equipment

CRUMB TEST

LAB INDEX: 56C-107 FEATURE: McGee Creek

BY: Calbow DATE: 2-27-81

NATURAL MOISTURE ☑ YES ☐ NO ☐ DISTILLED WATER ADDED

MOISTURE CONTENT 23.4 %

TEST WATER: EC = 21x/mho μsiemens/cm, pH = 7.0

WATER TEMPERATURE: INITIAL = 22 °C

3-2-81

SAMPLE NO.	DISH NO.	2 MIN.		15 MIN.		1 HOUR		24 HOURS		COMMENTS INCLUDE EC AND pH
		GRADE	TEMP.	GRADE	TEMP.	GRADE	TEMP.	GRADE	TEMP.	
		2	21°C	3	22°C	3	21°C	4	20.5°C	19x1 pH 7.0 11 mho

Figure 6L-1. - Crumb test (example data sheet).

CRUMB TEST

LAB INDEX: _____ FEATURE: _____

BY: _____ DATE: _____

NATURAL MOISTURE ☐ YES ☐ NO ☐ DISTILLED WATER ADDED

MOISTURE CONTENT _____ %

TEST WATER: EC = _____ μsiemens/cm, pH = _____

WATER TEMPERATURE: INITIAL = _____ °C

SAMPLE NO.	DISH NO.	2 MIN.		15 MIN.		1 HOUR		24 HOURS		COMMENTS INCLUDE EC AND pH
		GRADE	TEMP.	GRADE	TEMP.	GRADE	TEMP.	GRADE	TEMP.	

Figure 6L-2. - Crumb test (blank data sheet).

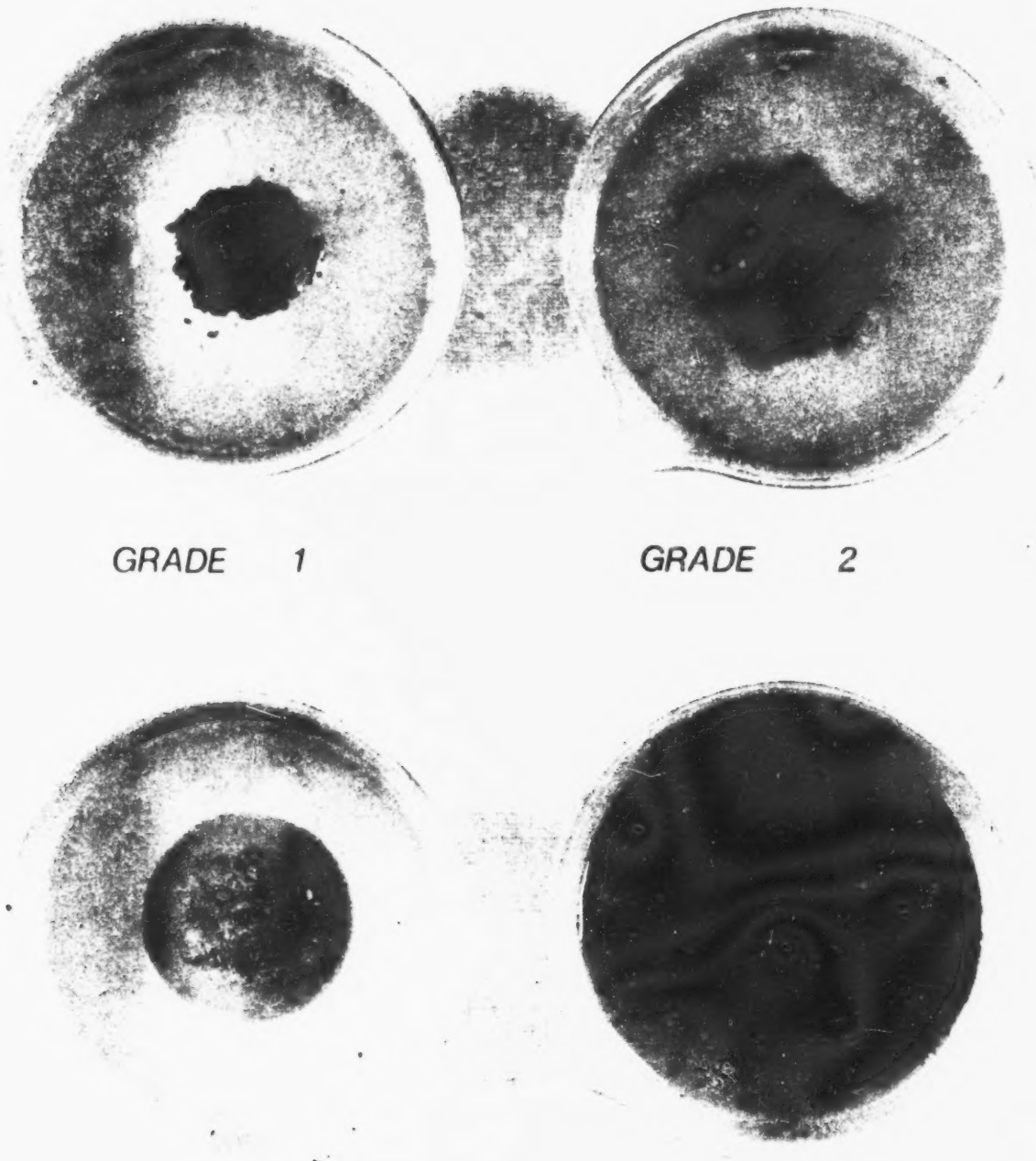

Figure 6L-3. - Soil crumb specimens and assigned dispersion grades.

APPENDIX M
FLOW RATE DATA SUMMARIES (ND3, ND2, and ND1 soils) ml/s

APPENDIX M - Continued
FLOW RATE DATA SUMMARIES (ND3, ND2, and ND1 soils) ml/s

Pinhole classification	50 mm head				180 mm head						380 mm head								
	0.00 0.20	0.21 0.40	0.41 0.60	0.61 0.80	0.41 0.60	0.61 0.80	0.81 1.00	1.01 1.10	1.11 1.20	1.21 1.40	0.81 1.00	1.01 1.20	1.21 1.40	1.41 1.60	1.61 1.80	1.81 2.00	2.01 2.20	2.21 2.40	2.41 2.60
ND1		XX				XX								XX					
ND1		XX				XX						XX							
ND1		XX			XX						X	X							
ND1		XX				XX						X	X						
ND1	X	X			X	X						X	X						
ND1		XX				X	X						XX						
ND1		XX				XX						XX							
ND1		X	X			X	X						X	X					
ND1		XX				XX							X	X					
ND1		XX				XX	X						XX						
ND1		XX				XX						XX							
ND1		XX				XX							XX						
ND1	XX						XX							XX					
ND1		XX				XX						XX							
ND1		XX				XX							XX						
ND1		XX				XX							XX						
ND1		X	X		X	X					X	X							
ND1		XX			XX						XX								
ND1		XX				XX							XX						
ND1	XX					XX							XX						
ND1		XX				XX							XX						
ND1		XX	X				XX							XX					
ND1		XX				XX							XX						
ND1		XX				XX						XX							
ND1		XX				XX							XX

APPENDIX M - Continued
FLOW RATE DATA SUMMARIES (ND3, ND2, and ND1 soils) ml/s

Pinhole classification	0.00 0.20	0.21 0.40	0.41 0.60	0.61 0.80	0.41 0.60	0.61 0.80	0.81 1.00	1.01 1.10	1.11 1.20	1.21 1.40	0.81 1.00	1.01 1.20	1.21 1.40	1.41 1.60	1.61 1.80	1.81 2.00	2.01 2.20	2.21 2.40	2.41 2.60
						100 mm head									380 mm head				
ND1		XX				XX						X	X						
ND1		XX				X	X						XX						
ND1	XX				X	X					XX								
ND1	XX					XX					XX								
ND2		XX					X	X							XX				
ND2		XX					XX							XX					
ND2		X	X						XX							XX			
ND2		XX					XX							X	X				
ND2		XX					XX							XX					
ND2		XX					XX								XX				
ND2	X		X						XX							XX			
ND2		X	X				XX									XX			
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CHEMICAL TESTS FOR DISPERSIVE SOILS - PROBLEMS AND RECENT RESEARCH

by

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January 1984

Applied Sciences Branch
Division of Research and Laboratory Services
Engineering and Research Center
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ABSTRACT

Research conducted at the Bureau of Reclamation, Denver, Colorado, has shown that the use of pore-water cation data to identify dispersive clay-soils, as suggested by Sherard, et al., is not a universal or accurate technique. Two hundred twenty-three soil samples from six different soil groups were analyzed according to the method of Sherard as well as analysis of variance and discriminant analysis. Results indicate that pore-water cation variables are poor dispersive/nondispersive discriminators for most of the soil groups investigated. The lack of agreement between pore-water analyses and pinhole test results may be attributed to the heterogeneity of soil, the qualitative nature of currently used pinhole test methodology, and the inappropriate use of a saturation extract in the pore-water method suggested by Sherard, et al.

INTRODUCTION

Because of the potential loss to life and property resulting from catastrophic failure of earthen structures constructed with dispersive clay-soils, there has been increasing interest in developing reliable physical and chemical tests to identify dispersive soils. Several physical tests [1, 2, 3]* are currently used to assess the dispersive nature of soils, however, these tests are qualitative and sometimes yield conflicting results on representative specimens of the same soil sample. Chemical tests on the soil saturation extract (or pore-water), which measure sodium (Na), potassium (K), calcium (Ca), and magnesium (Mg), are often used to corroborate physical test results on suspected dispersive soils. The use of chemical tests is based on the assumption that there is a chemical basis for the observed phenomenon of dispersion observed in clay-soils.

Sherard and co-workers [4] proposed a technique (hereinafter referred to as the "currently accepted method") where pore-water cation data are plotted on a percent sodium (%Na) versus TDS (total dissolved solids, in milliequivalents per liter) control chart (fig. 7-1). TDS was defined as the sum, in meq/L, of the cations (Na+K+Ca+Mg) present in the pore water. (Throughout the remainder of this paper "TDS" will be replaced by the more accurate term "Σcations"). In developing the chart on figure 7-1, pinhole test [3] results were used to classify the soils as D (dispersive) and N (nondispersive), and the boundaries seen on figure 7-1 were qualitatively determined by plotting pore-water data from samples that were also subjected to pinhole testing. Zone A corresponds to D behavior, and zone B corresponds to N behavior. Zone C may be considered a transition region containing both N and D soils.

Unfortunately, the currently accepted method did not satisfactorily corroborate the results of pinhole tests during dispersive testing programs conducted by the Bureau of Reclamation. Investigations were initiated to determine how well the currently accepted method relates to physical test results for soils from the McGee Creek Project, an earthfill dam under construction near Farris, Oklahoma. In addition, previously analyzed soil data, obtained from the SCS (Soil Conservation Service), U.S. Department of Agriculture, Lincoln, Nebraska, were also included in these investigations. Besides the documentation of the success or failure of the currently accepted method, these studies were also intended to suggest further research that might improve chemical and/or physiochemical techniques for identifying dispersive clay soils.

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* Numbers in brackets refer to listings in the Bibliography.

Table 7-1. - Percent correctly classified as nondispersive (N), dispersive (D), and total (T) for individual soil groups using the %Na versus Σcations chart

Group	Percent correctly classified		
	Nondispersive (N)	Dispersive (D)	Total (T)
TEST	35.7	81.8	60.7
SCSTQW	60.0	50.0	55.6
SCS74F	66.7	72.7	71.0
SCS75W	87.5	87.5	87.5
SCS76W	56.3	31.6	42.9
SCS77W	41.2	71.4	55.3
ALLDATA	56.7	68.9	62.8

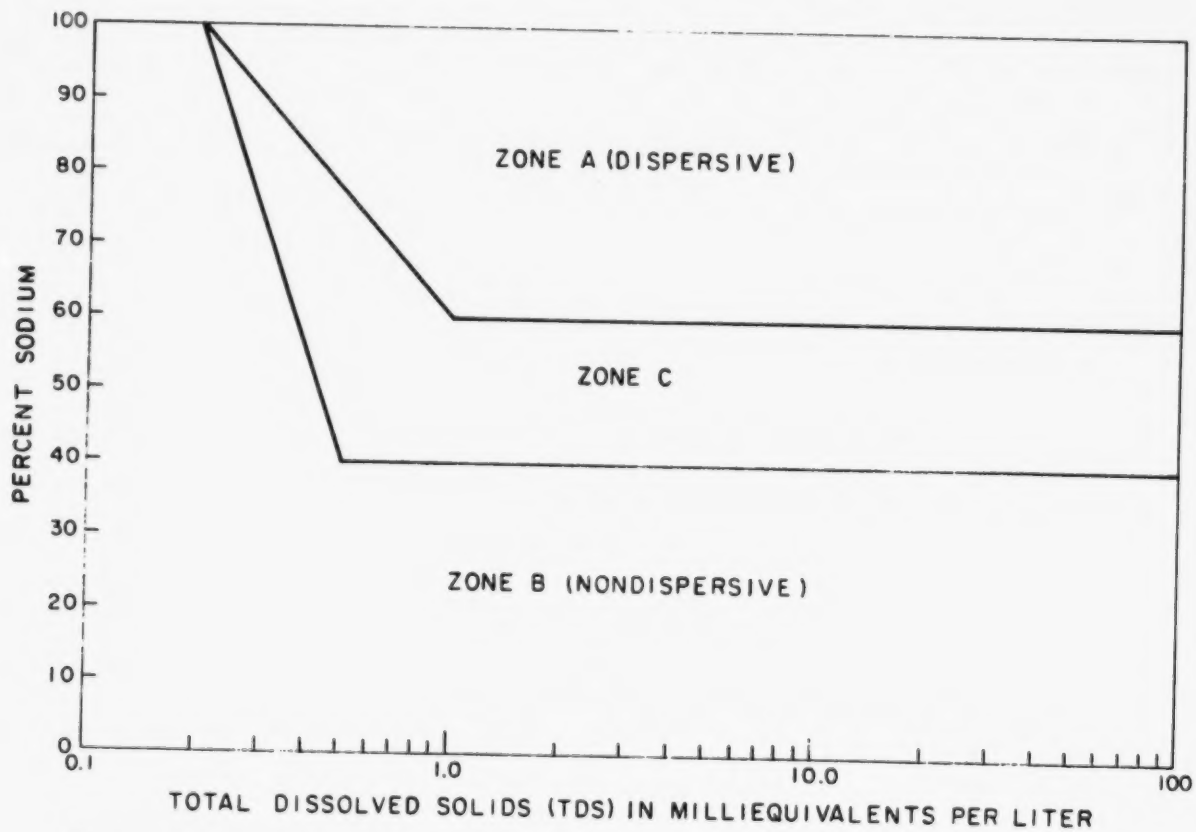

Figure 7-1. - %Na versus Σcations chart proposed by Sherard, et al. to identify dispersive clay-soils.

METHODOLOGY

Thirty-seven soil samples from the McGee Creek Project (hereinafter referred to as the TEST soil group) were processed for physical and chemical tests according to standard soil preparation methods. Fourteen of these samples were prepared in replicate, resulting in 61 total samples. These representative samples were blended and split into fractions for the pinhole test and pore-water chemical analyses. USDA Agricultural Handbook No. 60 [5] methods were used in preparing the saturation extracts with the exception that pore water was separated from the saturated paste using centrifugation [6]. Pore-water chemical analyses on replicate samples were used to estimate the error between representative samples, and within-sample error was estimated by repeated chemical determinations on a composite saturation extract. Pinhole tests were performed according to the methods summarized by Kinney [7]. Details of the chemical analysis techniques are covered in another paper [8].

Pore-water cation data from the 61 TEST soil group samples and 162 other soil samples, analyzed and classified by the SCS Laboratory, Lincoln, Nebraska, were evaluated in this study. The SCS data were from five different locations, so a total of six different soil data sets were evaluated. Pinhole test results were used to classify the samples as D or N, and samples classified as intermediate were excluded from the data sets. SCS values for meq/L of Na and Ca+Mg were calculated from %Na and Σcations data. For these data, the quantity (Ca+Mg) also includes K, although K concentrations are usually insignificant compared with typical pore-water Ca+Mg concentrations.

The 223 samples in this study were evaluated using the currently accepted method as well as univariate, one-way ANOVA (analysis of variance), and discriminant analysis. All three techniques were applied to individual soil data sets and the combined data set (ALLDATA). ANOVA was performed using a Hewlett-Packard HP-85 desktop computer with the HP General Statistics Package software, and discriminant analysis was performed using a CYBER-70 version of the SPSS (Statistical Package for the Social Sciences) subprogram DISCRIMINANT [9].

Univariate, one-way ANOVA was used to test individual pore-water variables for each soil group to determine whether statistically significant differences exist between N and D means for the tested variable. Discriminant analysis, a multivariate statistical classification (or pattern recognition) technique, derives a special classification equation, called the discriminant function. This function is a linear combination of the variables entered into the analysis and is of the following form:

$$D_i = d_1V_1 + d_2V_2 + \cdots + d_iV_i + C_i$$

where D_i is the discriminant score (calculated for each sample or case); d_i's are weighting coefficients; V_i's are raw data corresponding to the variables entered into the analysis; and C_i is a derived constant. If a given set or subset of variables acts to discriminate between N and D soils, the discriminant scores for the N and D groups will be significantly different.

In this way, discriminant analysis provides a statistical test to evaluate whether a particular subset of pore-water variables act in combination to distinguish between N and D soils.

Details regarding the specific variable subsets analyzed and SPSS subprogram DISCRIMINANT are covered elsewhere [8, 9, 10].

RESULTS AND DISCUSSION

Pore-water cation data for TEST and five SCS soil groups are plotted on %Na versus Σcation charts on figure 7-2. The percentage correctly classified for each soil group according to N, D, and T (total) using the %Na versus Σcations chart may be seen in table 7-1. Percent correctly classified is defined as the percentage of N or D samples whose pore-water data are plotted in the correct (corresponding to the physical test classification) zone on the %Na versus Σcations chart. Samples which plotted in Zone C were counted as misclassifications. Note the wide differences between groups for the percent correctly classified. Only one group, SCS75W, indicated reasonable success (87.5 percent correct), while three groups, SCSTQW, SCS76W, and SCS77W, showed less than 60 percent total correctly classified.

Table 7-2 indicates the within-sample error based on replicate measurements of Na and Ca+Mg on a composite TEST saturation extract. It shows that within-sample error is small compared to error between replicate samples. This is graphically displayed for TEST representative sample replicates on figure 7-3. Note that the log scale for Σcations greatly exaggerates the 95 percent confidence intervals for soils with low total cation concentrations. These data point out the variability in pore-water cation data resulting from the heterogeneous nature of soils.

The results of a one-way method of classification, ANOVA, performed on independent, dependent, and log-transformed variables for each soil group may be found in table 7-3. The treatments were dispersive and nondispersive, and table 7-3 lists the F-ratios associated with each variable. Underlined F-ratios indicate that the N and D means for the given variable are significantly different at the 95 percent confidence interval (α=0.05). ANOVA was used to determine if there was a statistical basis for using pore-water cation data to identify N and D soils. It can be assumed that a variable having a low F-ratio would provide little discriminating ability. Table 7-3 points to the important observation that each soil group has a different set of F-ratios for the variables used in this study. It is also important to note that the F-ratios seen in table 7-3 seem to corroborate the results from the currently accepted method as well as the success or failure of discriminant analysis. The SCS75W exhibits significant F-ratios for all variables, and this group also showed reasonable classification success with the %Na versus Σcations chart. TEST soils do not conform to the boundaries for N and D suggested by the currently accepted method, however, significant F-ratios for %Na and Σcations indicate that the currently accepted method could be used with modified N and D decision boundaries. Visual inspection of chart a on figure 7-2 confirms this observation.

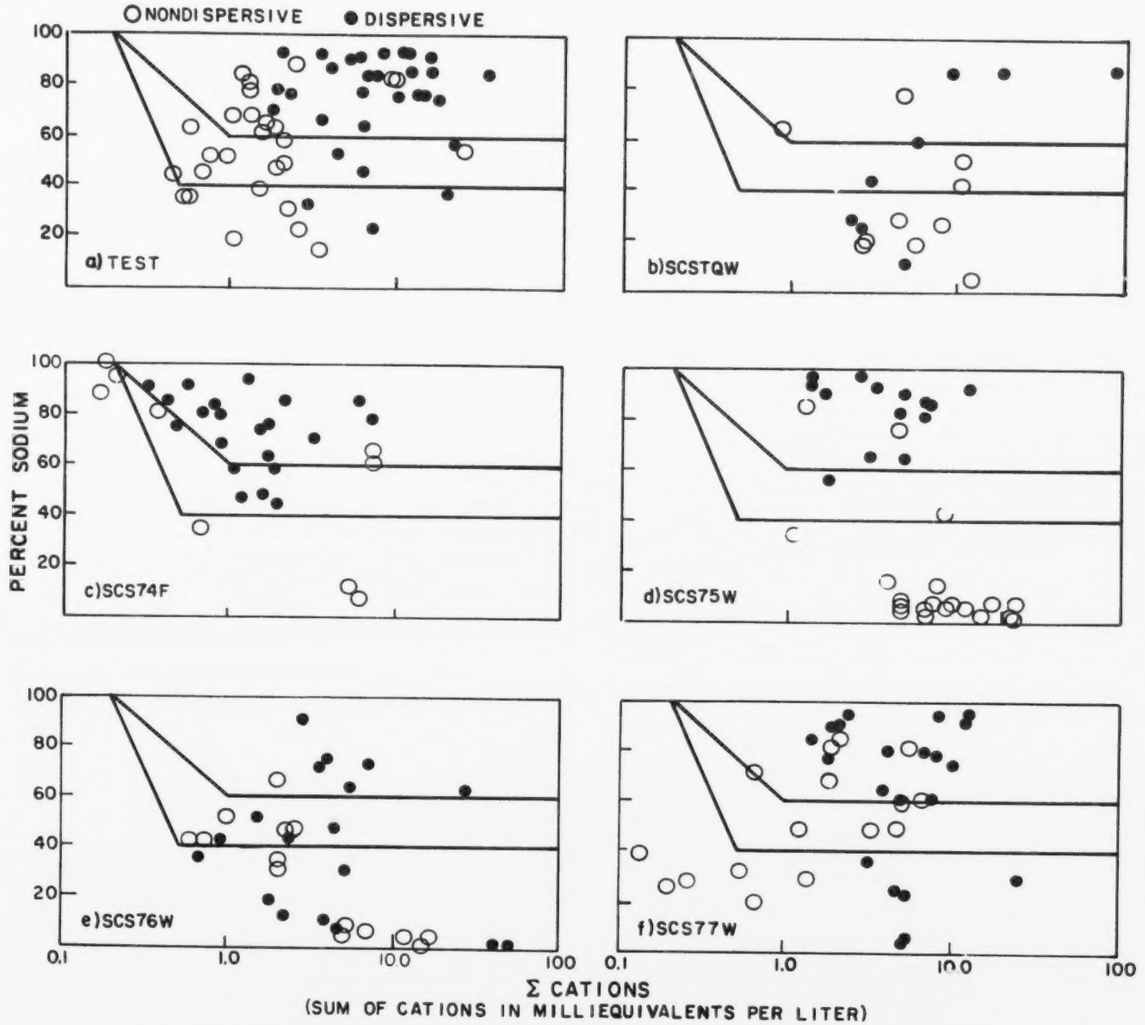

Figure 7-2. - Pore-water data plotted on %Na versus Σcations charts.

Table 7-2. - Within sample error for replicate chemical analyses

Parameter	Mean	Standard deviation	95% confidence limit	% relative standard deviation
Ca+Mg (meq/L)	4.73	0.091	0.178	1.92
Na (meq/L)	10.9	0.017	0.033	0.16
K (meq/L)	0.23	0.001	0.002	0.43
Σcations (meq/L)	15.8	0.086	0.169	0.55
%Na	68.6	0.381	0.740	0.55
SAR*	7.06	0.065	0.127	0.92

* SAR - Sodium adsorption ratio.

Table 7-3. - Summary of a one-way method of classification (ANOVA) using nondispersive (N) and dispersive (D) soils as the test treatments. Underlined values indicate a significant difference between N and D at the $\alpha = 0.05$ level

Group	Variables tested and associated F-ratios							
	%Na	Σcations	Na	Ca+Mg	SAR*	log (Σcations)	log(Na)	log (Ca+Mg)
TEST	29.6	20.4	25.8	2.90	40.5	67.5	89.5	8.70
SCSTQW	1.95	1.26	1.87	0.386	3.00	0.640	1.57	0.125
SCS74F	2.23	1.94	0.020	7.70	2.05	0.176	2.82	0.713
SCS75W	98.0	9.81	20.8	25.3	45.8	10.7	31.6	50.1
SCS76W	3.24	0.994	0.097	1.17	1.74	0.441	1.26	1.41
SCS77W	1.78	10.2	8.71	3.28	7.65	19.6	13.0	5.06

* SAR - Sodium adsorption ratio.

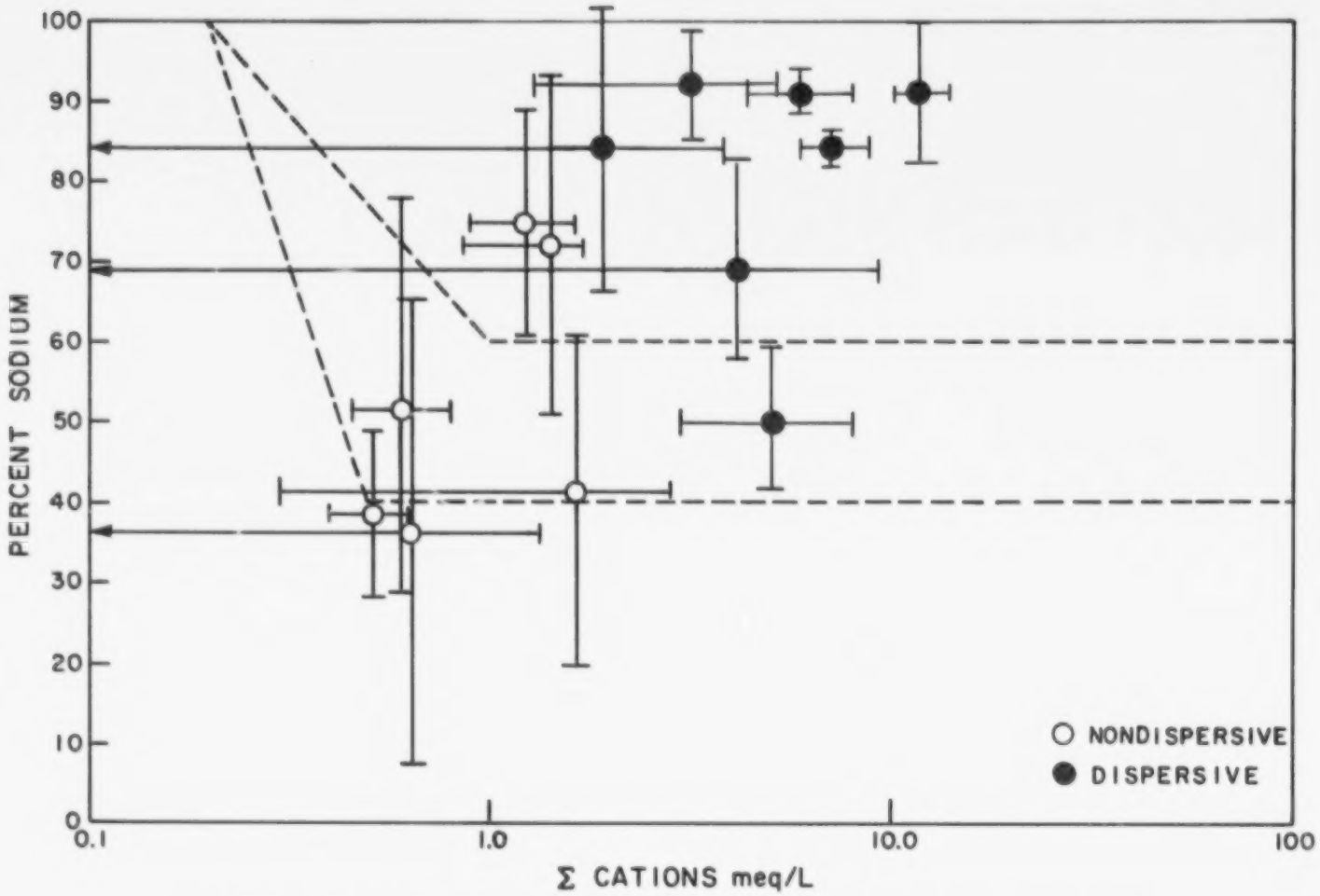

Figure 7-3. - Means and 95% confidence intervals for TEST pore water data.

Discriminant analyis results appear in tables 7-4 and 7-5. These tables list the standardized discriminant function coefficients associated with the particular variables included in the analysis, eigenvalues, and percent correctly classified for each soil group. Standardized discriminant functions contain no constant and the variable-associated coefficients correspond to the relative discriminating power of each variable. The eigenvalue is a diagnostic parameter which indicates the success of a given discriminant function is in distinguishing between N and D soils. In this study, an eigenvalue greater than 3 would be considered "good."

Table 7-4 lists the standardized coefficients and eigenvalues for discriminant analyses performed with %Na and log Σcations (the variables used in the currently accepted method) as the analysis variable subset. Note that only two soil groups, SCS75W and TEST, indicate even moderately acceptable eigenvalues, and that each soil group exhibits unique standardized discriminant function coefficients. The data in table 7-4 corroborate the results obtained from the currently accepted method (fig. 7-3) and the ANOVA results (table 7-3).

Table 7-5 lists the discriminant analysis results for the best observed discriminant function for each soil group using pore-water variables and their log transformations. While three soil groups (TEST, SCS74F, SCS75W) exhibit reasonably good percent correctly classified data, none of the eigenvalues indicate functions with good separation between N and D groups. The all-groups histogram for the best SCS74F discriminant function (eigenvalue equals 2.92) may be seen on figure 7-4. Histograms are a graphic, visual means of judging a successful discriminant function. Ideally, a successful discriminant function will have a histogram with separate and distinct distributions for N and D groups, and the two distributions will be widely separated with no overlap between distributions. The SCS74F histogram shows two distinct distributions, however, there is little separation between N and D, and some overlap between groups. Figure 7-5 is the all-groups histogram for the best ALLDATA discriminant function (eigenvalue equals 0.449), which points out the obvious significant overlap between N and D groups.

The reason that all pore-water variables and their tranformations were used was to give the benefit of the doubt to the currently accepted method and the general hypothesis that pore-water variables can discriminate between N and D soils. Data presented here indicate that pore-water cation data do not discriminate between N and D soils.

Since pore-water cation data alone do not provide adequate N and D classification, a 30-sample subset of the TEST data, which included weight percent water at saturation (%H_2O), conductivity (EC) in microsiemens corrected to 25 °C, and pH, was subjected to discriminant analysis. Method DIRECT was performed using %H_2O, pH, log(EC), and log(Na) as the variable subset. The discriminant function derived from these variables was found to be successful, having an eigenvalue of 3.53 (the highest value encountered in this study). These results may be visually verified by inspecting the all-groups histogram for this function, found on figure 7-6. Note the separation between group centroids and the lack of overlap. These data strongly suggest that other pore-water variables besides cations may have discriminating ability for certain soils.

Table 7-4. - Standardized discriminant function coefficients, associated eigenvalue, and percent correctly classified for individual soil groups using dependent variables, %Na and log(Σcations)

Group	%Na	log (Σcations)	Eigen-value	Percent correct		
				N	D	Total
TEST	0.519	0.852	1.56	92.9	87.9	90.2
SCSTQW	0.892	0.237	0.128	60.0	50.0	55.6
SCS74F	1.18	0.828	0.147	55.6	68.2	64.5
SCS75W	0.994	-0.018	2.58	91.7	93.8	92.5
SCS76W	1.00	0.009	0.098	56.3	63.2	60.0
SCS77W	0.181	0.962	0.561	64.7	85.7	76.3
ALLDATA	1.01	0.576	0.404	72.1	72.3	72.2

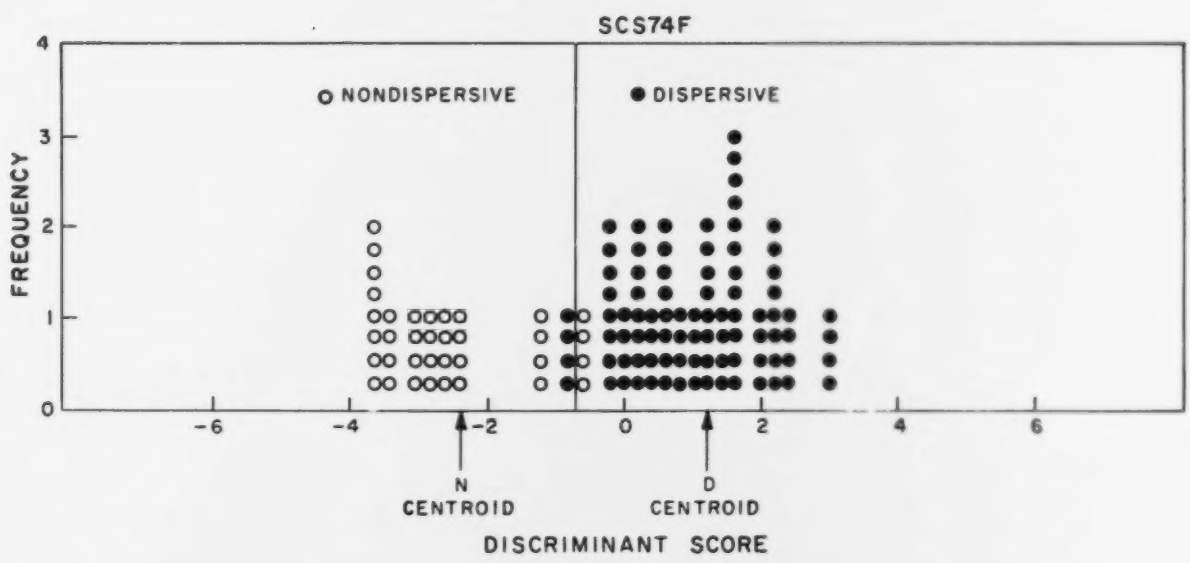

Figure 7-4. - All-groups histogram for the best discriminant function derived for soil group SCS74F pore-water data.

Table 7-5. - Standardized discriminant function coefficients for best discriminant analysis on each soil group

Group	%Na	Σcations	Na	Ca+Mg	SAR*	Log (Σcations)	Log (Na)	Log (Ca+Mg)	Eigen-value	Percent correct		
										N	D	Total
TEST	1.38	1.45	-0.649	-	0.133	0.240	-2.78	0.887	1.96	92.9	87.9	90.2
SCSTQW	-	-	-1.20	0.997	-	-	-	-	0.321	80.0	87.5	83.3
SCS74F	4.46	-7.94	4.82	-	-1.44	12.8	-7.24	0.566	2.92	88.9	100.0	96.8
SCS75W	1.58	0.145	-0.333	-	0.864	-1.26	-0.182	1.76	2.73	91.7	93.8	92.5
SCS76W	3.08	-2.35	5.16	-	-5.02	9.44	-3.72	-5.75	0.467	75.0	78.9	77.1
SCS77W	-3.19	-0.687	2.29	-	-3.41	8.02	-0.558	-8.05	1.07	82.4	81.0	81.6
ALLDATA	-0.721	0.117	0.368	-	-0.447	0.031	-0.368	-0.433	0.449	75.0	72.3	73.5

* SAR - Sodium adsorption ratio.

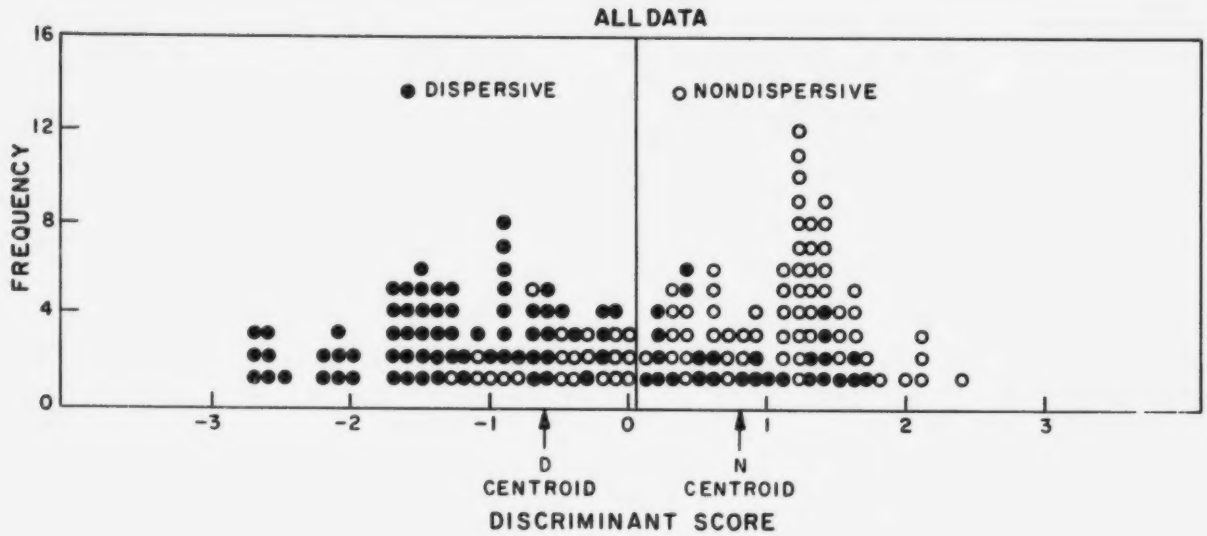

Figure 7-5. - All-groups histogram for the best discriminant function derived for the ALLDATA data set. Note the extensive overlap and lack of separation between N and D samples.

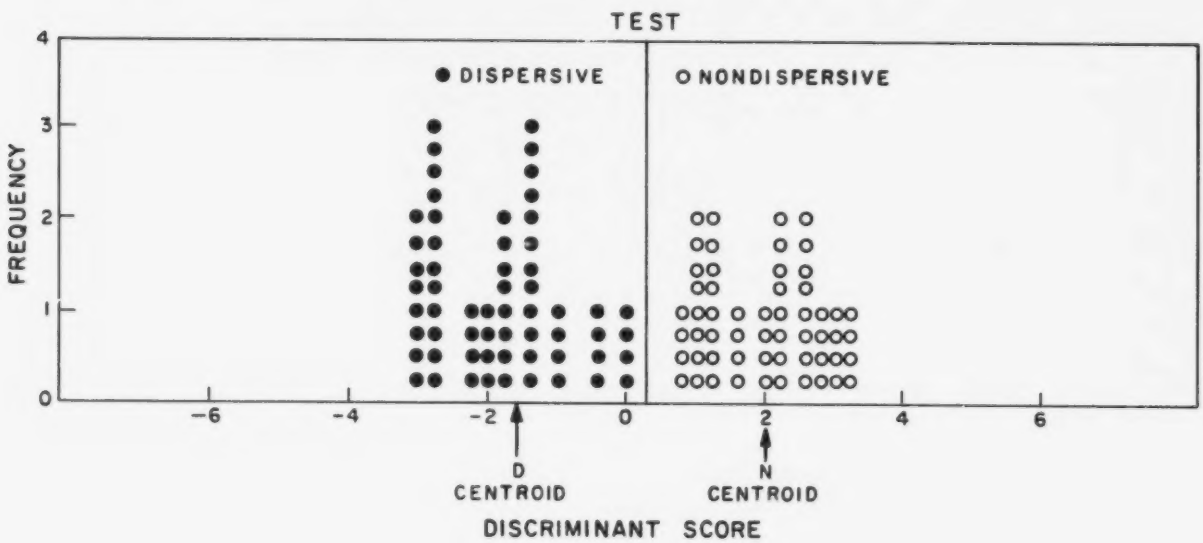

Figure 7-6. - All-groups histogram for the TEST discriminant function derived using $\%H_2O$, pH, log(EC), and log(Na) as the analysis variables.

The success of the TEST discriminant function, which included pH, EC, and weight percent H_2O at saturation, suggested that for certain soils discriminant analysis could be used to classify samples not originally used to derive the discriminant function. Procedures for using simple field test procedures with discriminant analysis are described by Craft [10]. It should be noted that not all soils will exhibit discriminating pore-water variables and thus the use of discriminant analysis for predicting N and D behavior is limited. One advantage of discriminant analysis on appropriate soil groups is that probabilities of group (N or D) membership are calculated for unknown samples. Membership probabilities allow for a more analytical method of accepting or rejecting test results. For example, a sample classified as N with a membership probability of 0.80 would probably be retested since there is a 1 in 5 chance that the assigned group (N) is wrong.

SUMMARY AND SUGGESTED RESEARCH

It has been shown that the currently accepted method does not support physical test results for five of the six soil groups investigated. There can be no doubt that the universal applicability claimed for the currently accepted method is simply not true. ANOVA and discriminant analysis results strongly support the idea that pore-water cation data may be used to classify only certain soils. Additionally, the inclusion of $\%H_2O$ and pH resulted in an improved discriminant function for TEST soils, indicating that other variables besides the pore-water cations may help identify the dispersive soils. Indeed, discriminant analysis may be used to classify unknown soil samples from soil groups with strongly discriminating pore-water variables.

The author feels that the observed lack of agreement between physical and chemical tests may be attributed to the following interrelated factors: the qualitative nature of physical tests, the heterogeneous nature of soil samples, and problems related to the use of the saturation extract in the currently accepted method.

Currently used physical tests represent a problem since qualitative results cannot be subjected to rigorous statistical analyses such as multiple regression techniques. Acciardi [11] has shown that the pinhole test, generally accepted as the most reliable physical test, may be slightly modified to provide quantitative data related to dispersion. Wider use of quantitative physical tests would offer the advantages of eliminating the uncertainty with engineering interpretation of qualitative data and provide an analytical basis for evaluating chemical techniques.

As previously mentioned, the heterogeneous nature of soils and the resulting error associated with replications between representative samples may pose serious problems for any N/D classification technique that relies on pore-water data. Even if reliable quantitative physical tests are used, the variability inherent in soil samples may reduce any quantitative predictability derived from chemical tests. It should also be noted that studies which paired replicate pinhole tests with a single pore-water analysis (as

done in developing the currently accepted method) may have drawn conclusions that replicate pore-water tests may have modified or ruled invalid. The variability seen on figure 7-3 further supports this point.

Finally, the use of a saturation extract in the currently accepted method may pose problems which have not been addressed by geotechnical researchers. Since a saturated soil paste is formed using a small volume of water compared to the soil sample weight, it is likely that the pore water will be saturated with respect to certain ions extracted from soil particles. Saturation, which may be caused by low volumes of water and/or the presence of high concentrations of absorbed ions, is only part of the problem. Recent unpublished work by Natusch and Skogerboe on water extractions of soil indicates that soil-associated salts are only partially dissolved with repeated leaching using 10:1 (water:soil) extraction ratios. These data suggest that sparingly soluble soil ions may not be quantitatively recovered even if high water:soil extraction ratios are used. Thus, it is likely that a saturation extract does not contain all ions which may be responsible for dispersion. While it is generally accepted that dispersion is caused by processes related to soil absorbed or entrained ions, very little research has been published relating an appropriate extraction technique to an accurate physiochemical mechanism for dispersion.

The author recommends that geotechnical researchers concentrate their efforts on studies that will identify those soil variables responsible for dispersion. Since the charge repulsion model is generally accepted as a likely mechanism for dispersion, the studies should examine the roles of mineralogy, particle size distribution, and soil-related cations, anions, and organic compounds in influencing dispersive behavior. Also, special attention should be given to the selection of an appropriate extraction technique that will accurately isolate the soil constituents causing dispersion. A complementary approach, suggested by Yong and Sethi [12], would be to evaluate electrokinetic phenomena (such as the zeta potential) related to the electrostatic charge on colloidal soil particles.

It is further recommended that persons involved with dispersive soil testing programs for engineering projects rely on the currently accepted physical tests as the primary means of classifying soil samples. If pore-water data are to be used to corroborate physical test results, it is strongly suggested that pore-water discriminating variables be evaluated on a site-specific basis using techniques outlined in this paper. Discriminant analysis, when properly applied, provides a powerful statistical tool for evaluating the best subset of discriminating pore-water variables, and it is suggested that geotechnical researchers consider its use.

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[8] Craft, C. D. and R. G. Acciardi, "Failure of Pore-Water Analyses for Dispersion," Journal of the Geotechnical Engineering Division, ASCE (in press), 1984.

[9] Nie, N. H., et al., "SPSS: Statistical Package for the Social Sciences," 2d ed. McGraw-Hill, New York, N.Y., pp. 434-467, 1975.

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ELECTRONIC MONITORING OF CEMENT PRESSURE GROUTING AT RIDGWAY DAM

by

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January 1984

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Engineering and Research Center
Bureau of Reclamation

Denver, Colorado

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ABSTRACT

Electronic monitoring of USBR (Bureau of Reclamation) cement pressure grouting operations has been proposed for a number of years, but has only recently been used in North America [2]*. Ridgway Dam, located in west central Colorado, is the first Bureau dam on which electronic monitoring has been used; this is thought to be the first use of such monitoring in the United States. Electronic monitoring equipment has been used extensively in the chemical grouting industry for some time [3]; so the technology, both electromechanical and electronic, has been readily available.

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* Numbers in brackets refer to entires in the Bibliography.

DAMSITE DESCRIPTION

Ridgway Dam is a 325-foot-high (above the lowest foundation elevation) central core earthfill dam, located 20 miles south of Montrose, Colorado, on the Uncompahgre River. The site stratigraphy consists of interbedded sandstone, siltstone, and mudstone of the Morrison Formation. The dam construction contract was divided into two stages. The stage 1 contractor excavated the deep valley alluvium, grouted the dam foundation, constructed the river outlet works, and refilled the cutoff trench with compacted embankment to the existing river level. Electronic monitoring of the grouting was not used during the first stage.

Stage 2 construction will excavate the remaining abutment areas, construct the spillway, construct a drainage and grouting tunnel in the left abutment, grout the foundation, and complete the embankment from river level to the crest elevation of 6886 feet. Figure 8-1 shows the dam's maximum section and the divisions of the construction stages. Figure 8-2 shows a profile along the dam centerline, locating the areas to be grouted. And Figure 8-3 shows an aerial photograph of the damsite.

GROUTING REQUIREMENTS AND ELECTRONIC MONITORING DEVELOPMENT

The second stage contract for Ridgway Dam involves both blanket grouting and curtain grouting. Blanket holes were drilled beneath the central core where it contacted the foundation on both the right and left abutments. Blanket holes were also used upstream and downstream of the core contact in the more pervious left abutment. The blanket holes were generally 30 feet deep, except downstream on the left abutment, where they were 50 feet deep. The blanket holes were placed on a 10-foot-square grid pattern and angled 30°into the abutment. The blanket holes beneath the core were drilled by rotary diamond coring methods, while all blanket holes outside the core contact areas were percussion drilled. Figure 8-4 shows a plan of the blanket grouting areas for the second-stage construction.

Curtain holes were to be grouted to depths of 150 feet on the right and left abutments. A double-line grout curtain, extending from station 7+00 to station 4+95 (10 feet above the dam's crest), was used on the right abutment. Downstream curtain holes were grouted first. Eighty-foot grout patterns were used on the curtains with primary, secondary, tertiary, and quaternary holes having depths of 150, 90,60, and 30 feet, respectively. The method of split spacing to closure was used.

On the left abutment, curtain holes are to be grouted from both the dam foundation surface and the drainage and grouting tunnel. Figure 8-2 illustrates these grout hole locations. Curtain holes drilled from the surface of the left abutment are to be located 15 feet upstream of the dam centerline and angled 15° into the abutment. Grout holes in the drainage and grouting tunnel are also angled 15° into the abutment, but are located only 1-foot upstream of the dam centerline.

The original specifications required electronically monitoring the flow and pressure using transducers and providing a paper tape readout system. These specifications read as follows:

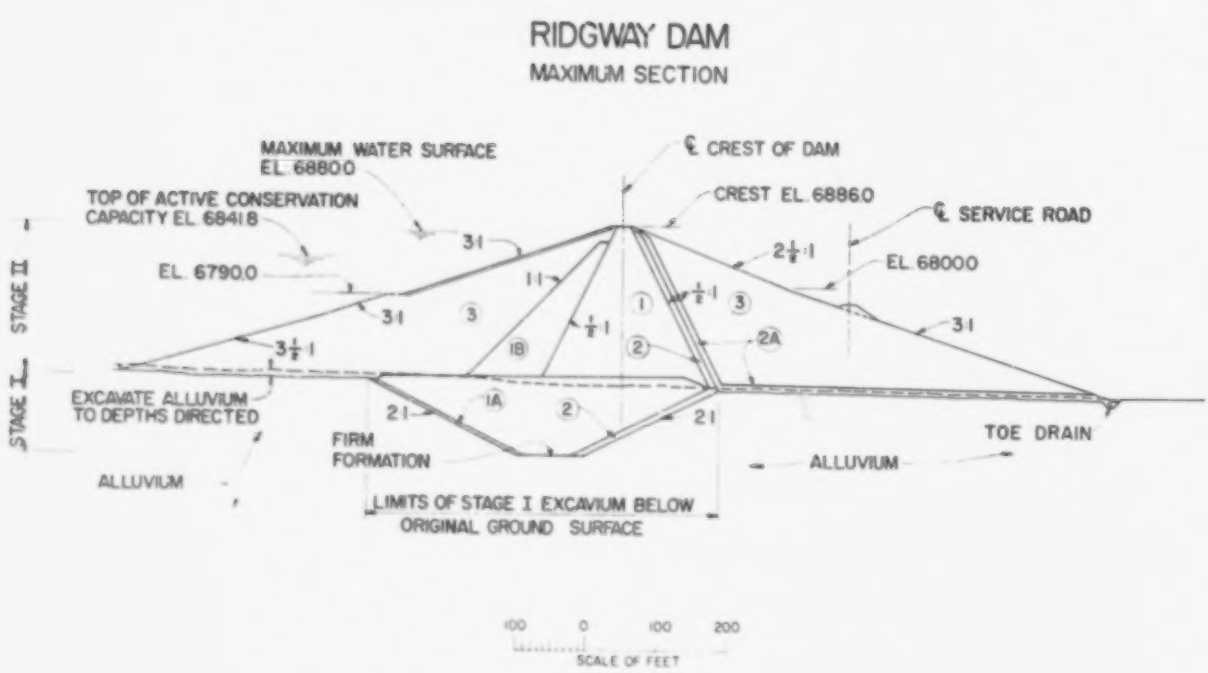


Figure 8-1. - Maximum cross section.

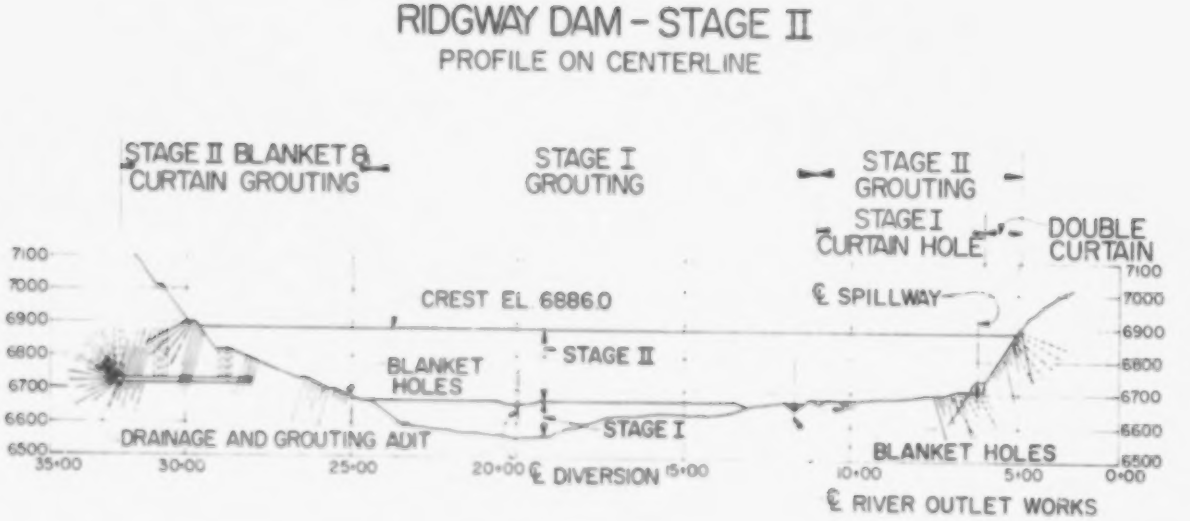


Figure 8-2. - Profile on centerline of dam.

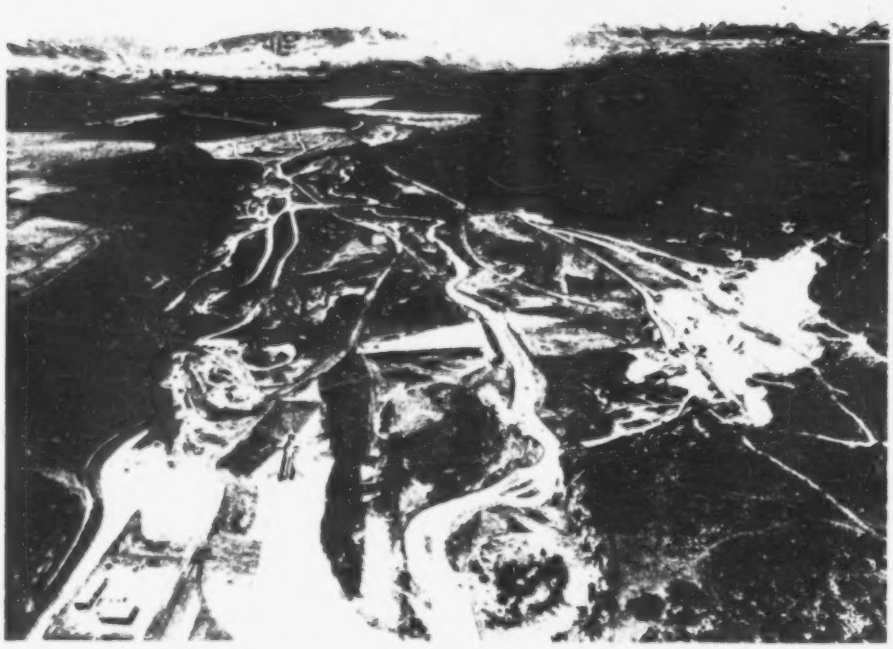


Figure 8-3. - Aerial view of damsite.

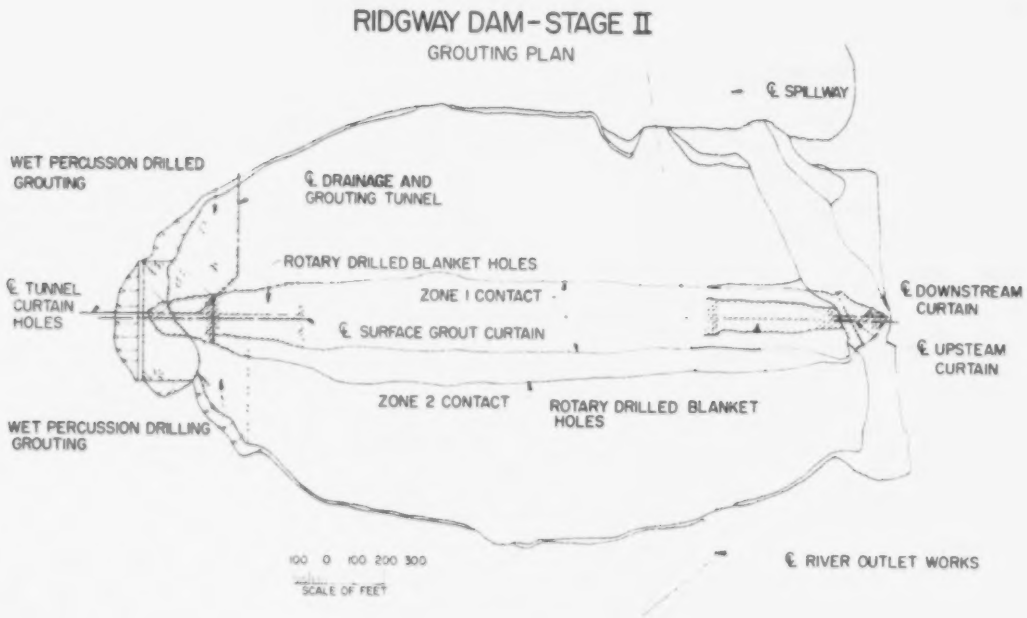


Figure 8-4. - Plan showing areas of blanket grouting.

"The Contractor shall provide an electrical flowmeter measuring device of an approved type for installation in the grout discharge line. The device shall have a continuous paper readout and shall be calibrated by the Contractor to accurately record the range of grout mixes used. The paper readout shall have sufficient capacity to record the entire flow rate variation for each stage. The Contractor will be required to recalibrate the device at the direction of the Contracting Officer. The Contractor shall submit the manufacturer's data concerning the instrument two weeks prior to its use.

"The Contractor shall also install an electrical transducer pressure recorder to measure the collar pressure at each hole. A continuous paper readout device shall be supplied and calibrated by the Contractor. The paper readout device shall have sufficient capacity to record the continuous pressure variations for each stage. If required, the Contractor shall construct a portable readout housing unit for the reading and recording of flowmeter and transducer readings."

The prime contract was awarded in the fall of 1982 to Granite Construction Company, and the grouting was subcontracted to HBC (Hayward-Baker Company) of Odenton, Maryland. In pregrouting talks with the Bureau, HBC proposed a much more comprehensive data collection and monitoring program than that specified, based on their experience with monitoring chemical grouting. The program involved using microprocessors, real-time video displays, and plotting equipment to calculate grout control parameters, plot critical items as a function of time, and provide hard copy final printouts. The system development was divided into two phases. Phase I would develop the system to monitor, record, and print real-time pressure grouting data. Phase II would develop the facilities to plot the plans, profiles, and contours of the completed grouted holes. To be available for the initial grouting of the blanket holes, microcomputer programs incorporating the Bureau's grouting requirements would be needed within 3 months. Internal Bureau discussions on whether to use the sophisticated electronic monitoring equipment were long and heated; but approval to develop the system was given early in 1983, based on the following assumed benefits:

- The ability to monitor additional holes more closely
- A reduction in the time required to make a determination of meeting refusal criteria and, thus, a reduction in the time needed to complete the grouting program
- Increased effectiveness of inspectors
- The potential for reducing the required inspection time
- The capability for quick and accurate plotting of stage-by-stage results
- The development of monitoring technology that will be available for future grouting work in the Bureau of Reclamation

Following approval of the phase I and II systems, computer programs were subcontracted to two different software development companies, and HBC began assembling the hardware and servomechanisms necessary to implement the system. The initial Bureau requirements for the system are listed in appendix A.

Phase I microcomputer hardware and software, pressure- and flow-monitoring equipment, and the grouting plant were available for the start of grouting in April 1983.

ELECTRONIC MONITORING EQUIPMENT

A schematic of the electronic monitoring grouting system is shown on figure 8-5. A centralized batch plant is used to facilitate the grouting. Figure 8-6 shows the centralized batch plant and the control center trailer, which houses the monitoring equipment. An eight-bit microcomputer using a FORTRAN 80 program and a CP/M operating system continually scans and updates the flow and pressure data for as many as five holes simultaneously. These data and the calculated control parameters are displayed on two video screens. One video screen is for the HBC personnel who input the initial drill, water test, and grout hole data and monitor the grouting operations from a house-trailer near the centralized batching plant. The other video screen is used by Bureau inspectors to complete the Bureau's traditional grouting forms. Figure 8-7 shows the phase I grouting arrangement inside the control trailer. Critical grouting information, comments, and notes are entered at the time of an event. Figure 8-8 shows a typical video screen display, and figure 8-14a shows the phase I equipment.

Pressure transducers on the grout header transmit collar pressure data to the control center. Flowmeter data are also transmitted to the control center. Header pressure adjustments were initially scheduled to be made by computer from the central control house, but the current practice has been for the contractor's personnel to manually control header pressures as directed by Government inspectors. The grout header arrangement is schematically shown on figure 8-9. A typical flowmeter is shown on figure 8-10.

Radio communication between the operations at the grout header and the control center, and between the control center and the batch plant are essential to facilitate smooth operations.

OUTPUT FROM MONITORING OPERATIONS

The grouting operations proceed as follows:

1. Hole locations are layed out and nipple holes (for blankets) are drilled to 4.5 feet.

2. Drill records for the nipple holes and the water test data are taken to the control center if grouting is necessary.

3. Packers are set at 1.0 foot depth and the nipple holes are grouted from 1.0 to 4.5 feet. The initial data are input by selecting the "start grouting hole" option, which is listed in the "menu" of the computer program. The list of "menu" options is shown on figure 8-11.

4. Each hole is drilled to its full depth if possible. Holes with 50-percent partial water losses are grouted at the time the water loss occurs.

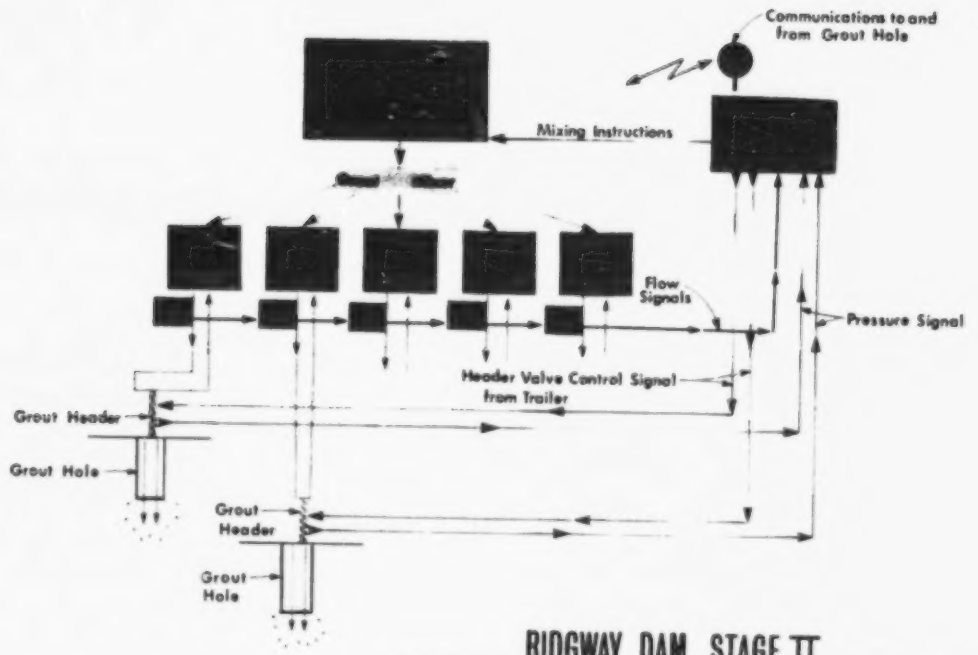


Figure 8-5. - Schematic of electronic monitoring system.

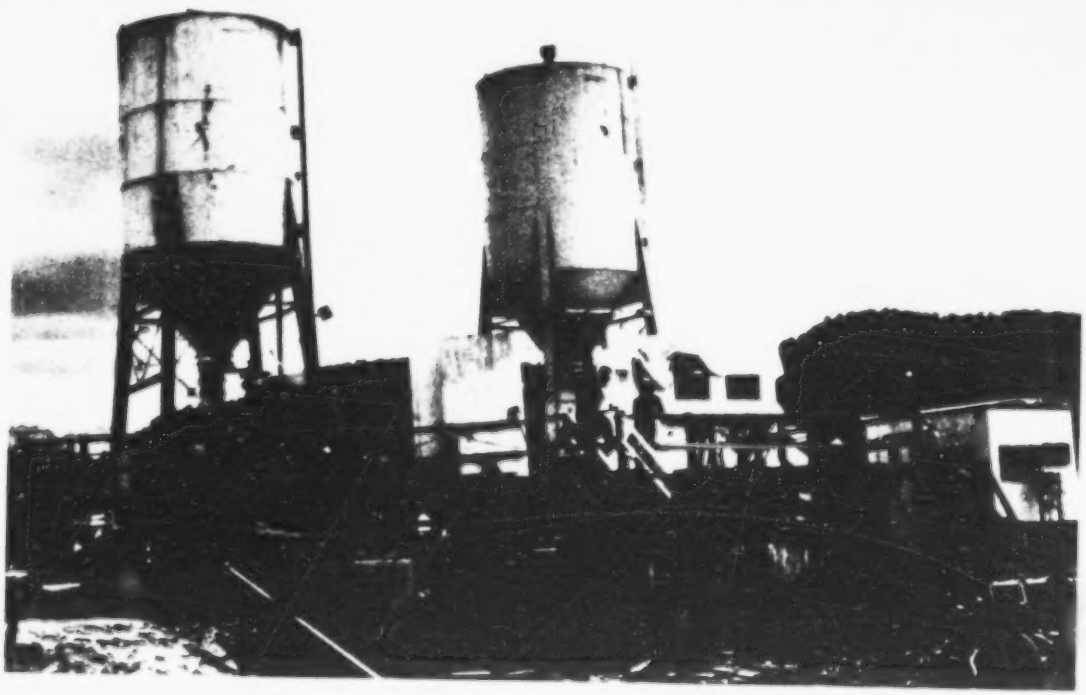


Figure 8-6. - Photograph of centralized batch plant and trailer housing the monitoring equipment.

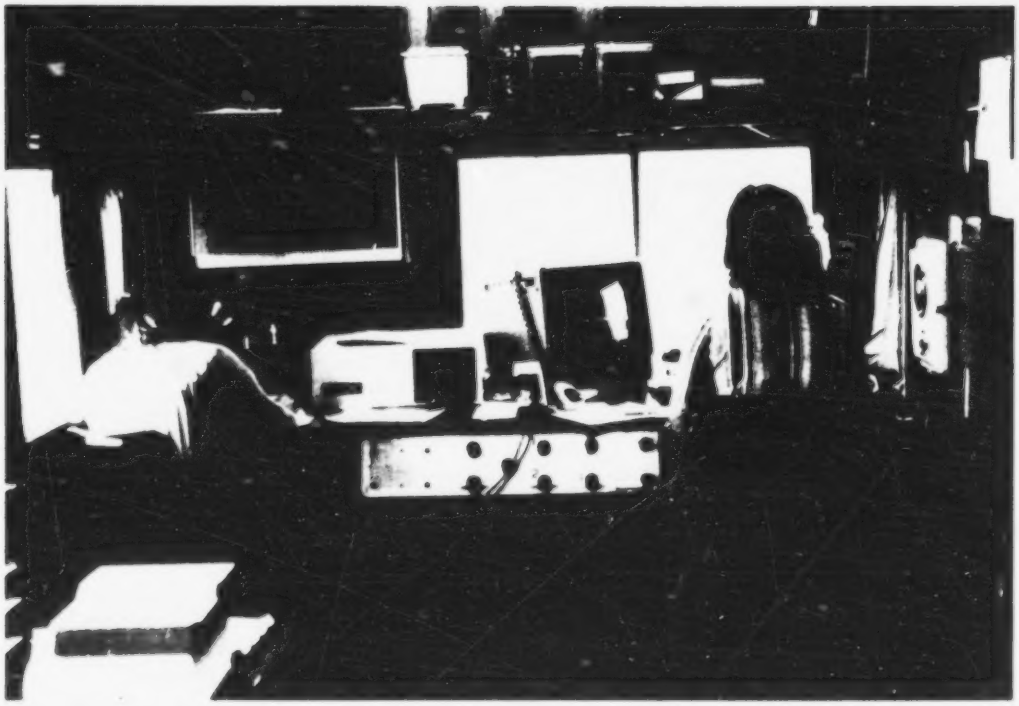


Figure 8-7. - Monitoring the grouting operations from a house-trailer at the centralized batching plant using phase I equipment.

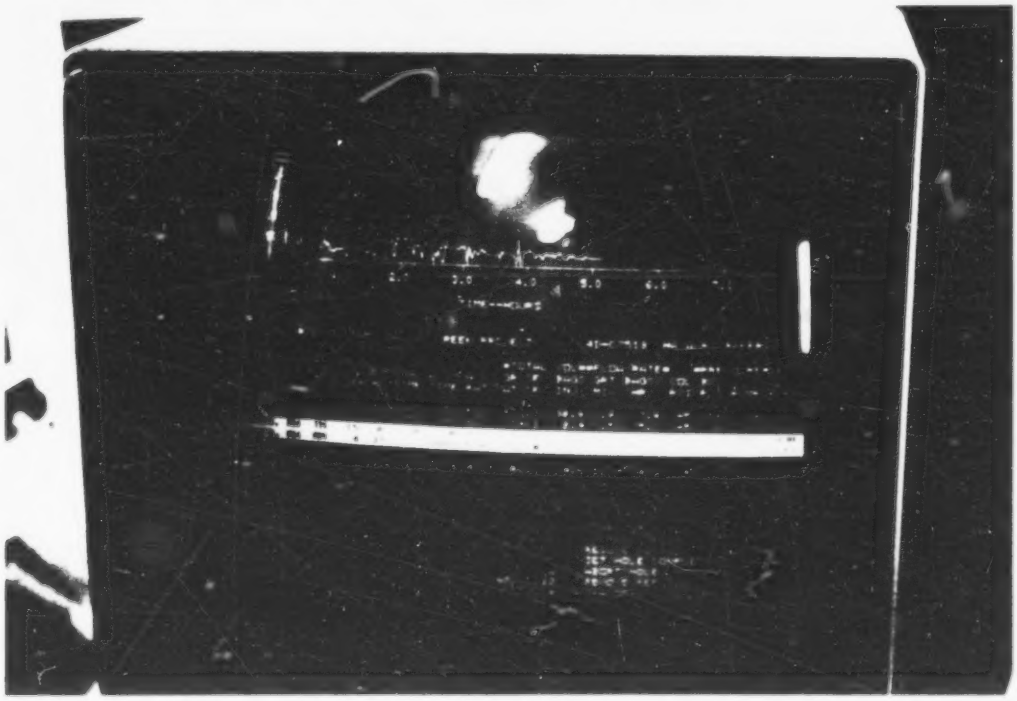


Figure 8-8. - Typical phase I video screen display.

GROUT HEADER FOR ELECTRONIC MONITORING

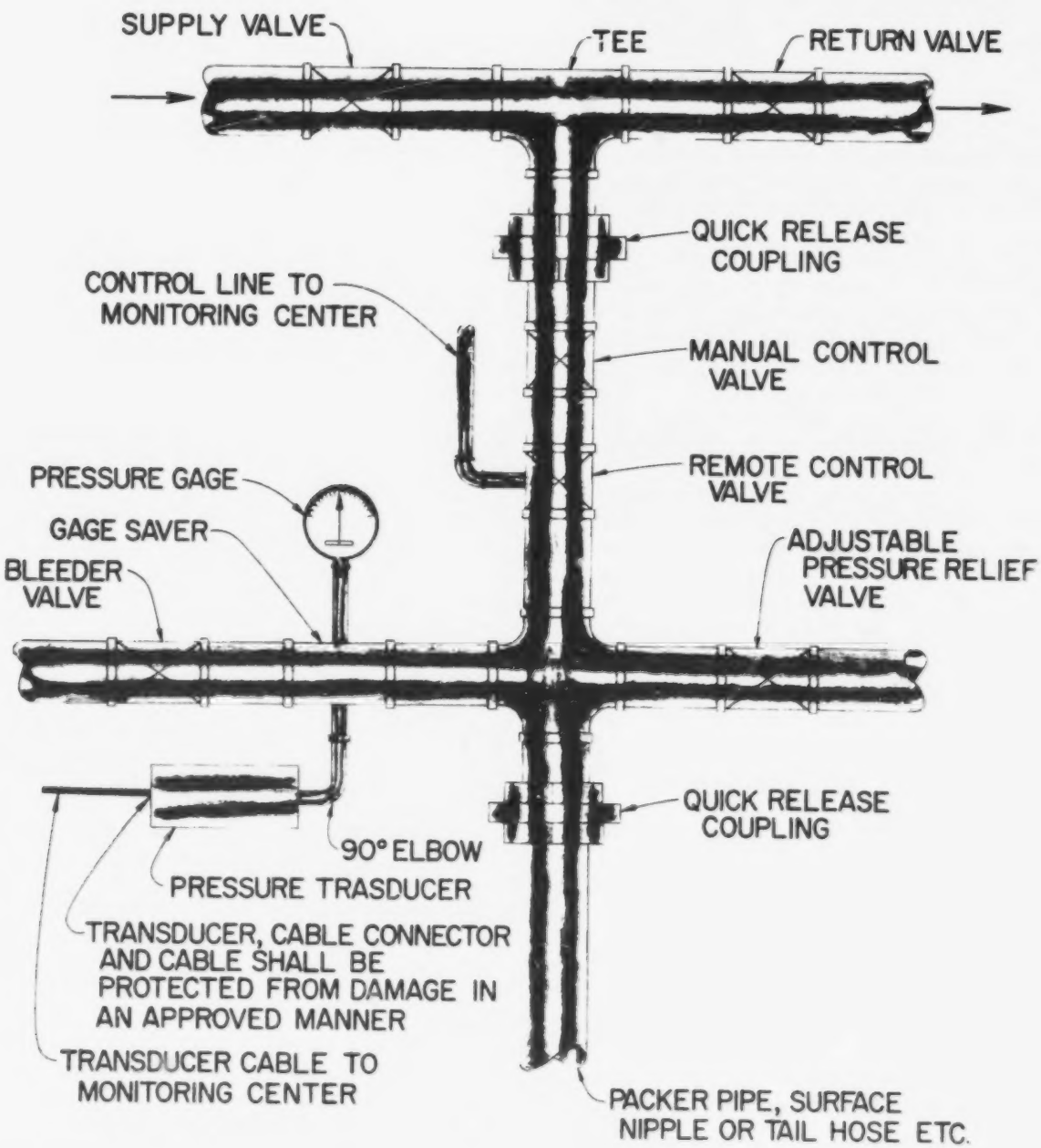


Figure 8-9. - Grout header.

5. The drill records are again taken to the control trailer.

6. The hole is water tested and stage grouted from the bottom to the top.

7. Following blanket hole grouting the curtain holes are drilled, water tested, and grouted.

Real-time phase I output from the monitoring equipment comes in three forms:

- Video display of real-time tabular data
- Video plotting of real-time graphical data
- Hard copy plots of real-time data

Prior to the start of grouting, all the specific hole data are input, as shown on figure 8-11. During grouting operations, changes in mix ratios are entered as "comments" at the time of their occurrence. The pressure and flow information from each hole is automatically updated for the stage being grouted. When each stage of a hole is complete, the following hard plots are made:

- Collar pressure versus time
- Flow rate versus time
- Total volume versus time
- Bags per hour versus time
- Flow rate divided by pressure versus time
- Total bags versus time

Figure 8-12 shows typical hard copy plots of these graphs.

Any of these graphs may be called up and plotted during real-time grouting. Upon completion of the stage grouting, a summary of the key data for this particular stage is printed as hard copy. Figure 8-13 shows a summary plot of data for the completed hole. All data are stored on diskettes for future reference. One diskette can hold the data for approximately 100 to 150 holes.

Phase II software and hardware were installed in late August 1983. The phase II system consists of a dot matrix printer, a pen plotter, a microcomputer, and a video display as seen on figure 8-14b. Diskettes from phase I are entered into the phase II system, corrected and the data stored in the 5 mega-byte hard disk. The phase II software program can then supply the following plots and reports.

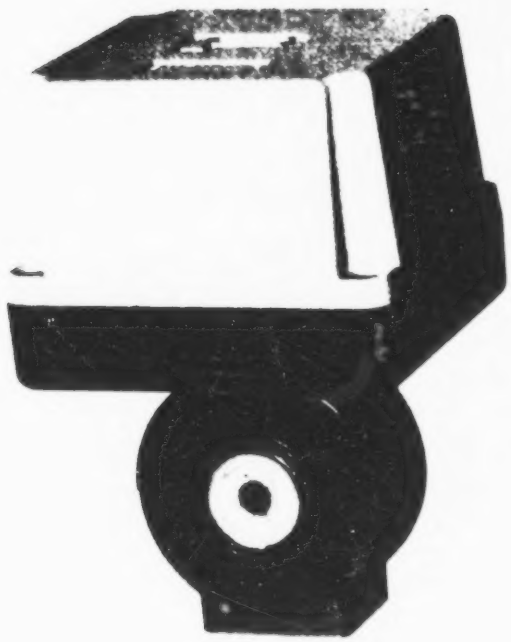
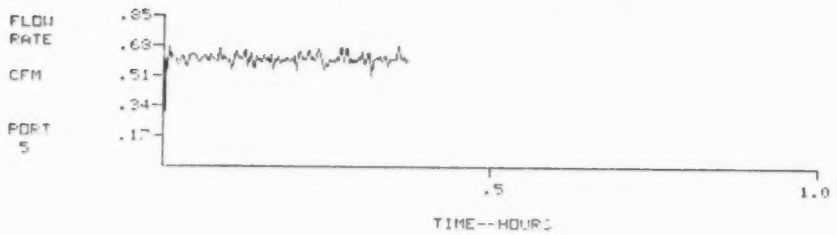


Figure 8-10. - Typical flow meter.




```
 18 15 10        RIDGWAY DAM  DALLAS CREEK PROJECT        4D-CT513  HAYWARD BAKER CO.
 28-03-83

                                                    *TOTAL VOL**FLOW RATE*    *PRESSURE*
     HOLE                    STAGE TOTAL W/C  W/C   GR/CF  BAGS GRT BAGS/ COL BOT
PT DESIGNATOR INC TEC  DEPTH  TIME TIME RAT  #/GAL   INJ  MIX  HR   PSI PSI PUMP P/P

 1
 2
 3
 4
 5 A24         RFR MRS   5- 30   23    8 1.0   14.4     2  .6   8.8   5  25      .833
 6
 7
 8


  SET UP WATER TEST.....1     SUSPEND HOLE.....8     SET ALARM THRESHOLD..15
  START GROUTING HOLE...2     RESUME HOLE.....9      ABORT WATER TEST.....16
  SET DATE/TIME.....3         SET HOLE COMPLETE.....10  STOP PROGRAM.....17
  COMMENTS ENTRY.....4        ABORT HOLE.....11      REMOVE REFUSAL.....18
  PLOT REAL TIME DATA...5     LIST HOLE STATUS.....12  LIST DRILL SUMMARY...19
  SCREEN DUMP HARDCOPY..6     ENTER DRILL SUMMARY...13  START WATER TEST.....20
  CHANGE W/C RATIO.....7      CHANGE HOLE STATUS....14  STOP WATER TEST.....21
SELECT: 22
```

Figure 8-11. - Typical input data for a hole before grouting and "menu" to select the required program option.

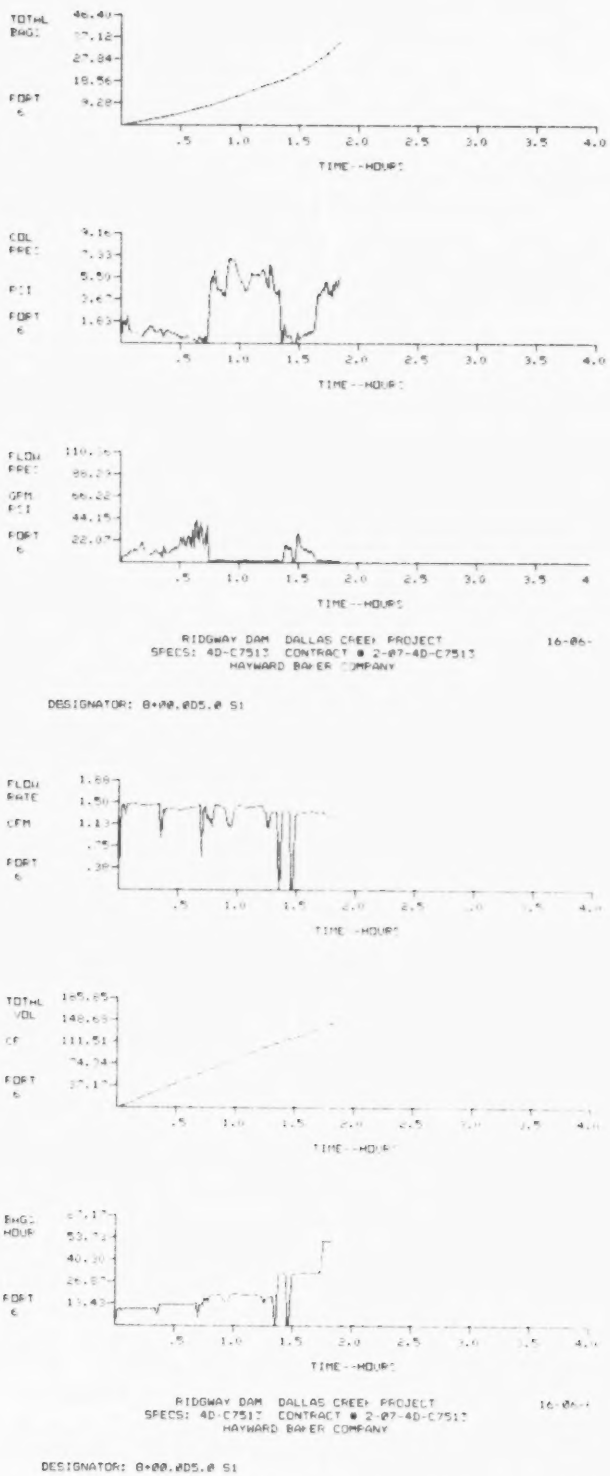


Figure 8-12. - Typical hardcopy plots drawn after each stage of grouting.

GROUT SUMMARY 11 33 40
RIDGWAY DAM DALLAS CREEK PROJECT 16-06-83
SPECS: 4D-C7513 CONTRACT # 2-07-4D-C7513
HAYWARD BAKER COMPANY

DESIGNATOR: 8+00.0D5.0 S1

STAGE DEPTH: 4- 15 PORT NO: 6

1. MAXIMUM VALUES ENCOUNTERED

FLOW RATE, CFM	1.5
BAGS/HOUR	53.7
FLOW/PRESSURE	88.291
INJ PRESSURE - COLLAR	7.3
INJ PRESSURE - BOTTOM	16.0

2. TOTALS

GROUT MIX INJECTED, CF	148.7
NUMBER OF BAGS	37.1
WATER, CF	130.5
TIME, MIN	112

3. AVG VOL (CF) GROUT INJ / LF 13.5

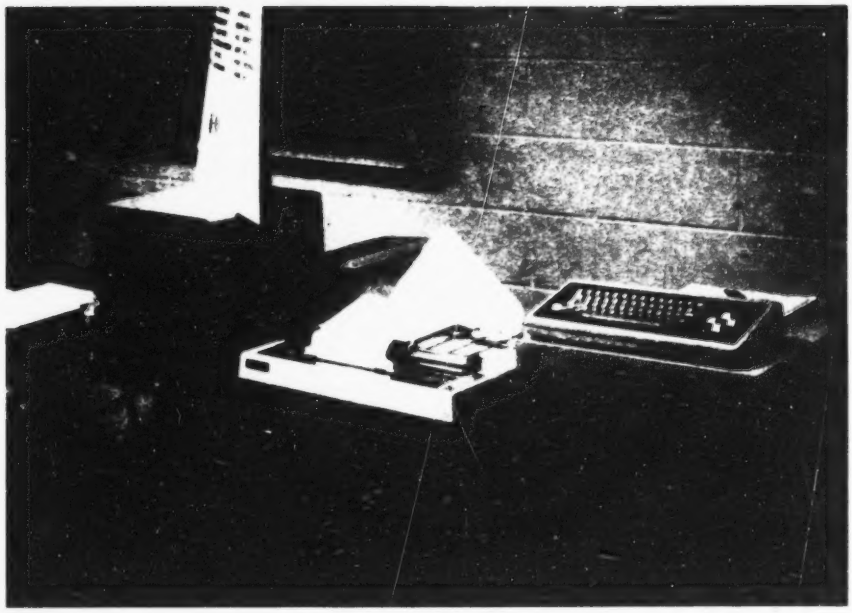
4. AVG BAGS INJ / LF 3.4

COMMENTS

TIME	COMMENT	OPR
0833	STARTED PUMPING AT 8:1	DJ
0843	COMM. TO NIP. 8+10.0U15.0	DJ
0851	SURF. LEAK 45'W	DJ
0856	STOPPED DUE TO WATER METER	DJ
0859	RESUMED PUMPING AT 8:1	DJ
0900	21 MIN AT 8:1,NOW 6:1	DJ
0904	ERROR REPORTED-IGNORE SURF.LK	DJ
0920	21 MIN AT 6:1, NOW AT 4:1	DJ
0958	12 MIN AT 4:1,24 MIN AT 2:1	DJ
1022	23 MIN AT 2:1, NOW 1:1	DJ
1030	USBR SET STAGE FIN: ZERO RET.	DJ

END OF COMMENTS

Figure 8-13. - Typical hardcopy plot of summary data completed after each stage of grouting.



(a) Phase I equipment.

(b) Phase II equipment.

Figure 8-14. - Phase I and phase II equipment.

Phase II reports:

1. Drill summary
2. Water testing summary
3. Grouting summary
4. Operations summary

Phase II plots (11-inch-wide):

1. Blanket hole plan
2. Blanket hole sections (longitudinal and transverse)
3. Centerline plots of curtain holes
4. Contour plots of:

   a. Collar elevations
   b. Water takes
   c. Grout takes

Figure 8-15 shows typical grout summary data, and figure 8-16 shows a typical plot of a longitudinal section through the blanket holes.

PRELIMINARY RESULTS

A preliminary evaluation of the electronic monitoring operations has been made, based on the partially completed grouting at Ridgway Dam. At this writing, only the right abutment blanket and right abutment curtain holes have been completed using electronic monitoring.

In general, the system allows the simultaneous monitoring of up to five grout holes. Originally the system was to use eight ports, six for grouting and two for water testing. However, grouting was limited to five holes with the sixth set of equipment available as a backup. Water testing through the last two ports was eliminated because of time restrictions.

It was originally planned to monitor the water testing in a manner similar to that of the grouting, but it proved to be too much of a problem to rapidly switch from water testing to grouting and then back, because the grout lines had to be flushed prior to the water test.

The control center personnel and Bureau inspectors monitoring the grouting quickly realized that two of the graphs seemed much more useful than the others: (1) flow versus time and (2) collar pressure versus time. These plots were used to determine when the hole was reaching refusal and when leaks and other problems had developed. Control center operators became so skilled that they would often predict surface grout leakage was occurring before it was seen by the header inspector.

The centralized location of the batch plant and the grout monitoring system have resulted in much better coordination between the Bureau and the subcontractor's grouting personnel. The use of hand-held radios, which allowed the contractor and Bureau personnel to communicate on the same radio frequency also improved this communication.

U S BUREAU OF RECLAMATION
DALLAS CREEK PROJECT
CONTRACT NO. 2-07-40-C7513
SPECS. NO. 40-C7513

RIDGWAY DAM, STAGE II
GROUT SUMMARY DATA - DATE: 20-10-83
SORTED BY LOCATION

DATE	TIME	PORT NO	DESIGNATION	IN-SPEC-TOR	TECH-NICIAN	LOCATION	STAGE DEPTH	MAX FLOW RATE	MAX PRESSURE COLLAR	MAX PRESSURE BOTTOM	MAX BAG RATE	MAX FLOW/ PRESSURE	TOTAL VOLUME MIX INJECTED
(D-M-Y)	(HHMMSS)						(FT)	(CF/MIN)	(PSI)		(BG/HR)	GPM/PSI)	(CU/FT)
27- 5-83	223800	6	5+79 0D25 0 S1	DPP	DBB	5+79.0- 25.0	0- 4	1.9	8	10	33.1	26.833	39.4
27- 5-83	150420	4	5+85 3D15 0 S1	DM	STO	5+85.3- 15.0	0- 4	.8	44	47	8.6	6.791	4.1
28- 5-83	022200	3	5+85 3D15 0 S1	DPP	DBB	5+85.3- 15.0	4- 15	3.4	2	5	79.5	9.433	36.9
27- 5-83	195300	3	6+00 0D25 0 S1	DCP	DBB	6+00.0- 25.0	0- 4	1.8	8	10	55.3	40.463	103.3
28- 5-83	200300	3	6+02 0D55 0 S1	DPP	DBB	6+02.0- 55.0	0- 4	2.4	1	3	92.2	24.062	96.8
31- 5-83	194300	3	6+50 0D85 0	TJN	DBB	6+50.0- 85.0	15- 30	1.5	14	33	24.8	76.135	319.8
28- 5-83	231100	3	6+50 0D65 0 S1	DP	DBB	6+50.0- 65.0	0- 4	2.3	4	6	91.3	107.633	33.6
27- 5-83	205110	3	6+52 0D25 0 S1	DCP	DBB	6+52.0- 25.0	0- 4	2.8	4	7	112.5	160.812	44.3
31- 5-83	202810	3	6+60 0D35 0	TC	RMR	6+60.0- 35.0	15- 30	1.8	12	29	7.2	1.178	1.3
1- 6-83	231100	3	6+60 0D35 0	TC	RV	6+60.0- 35.0	4- 15	3.4	0	8	76.3	163.281	333.2
27- 5-83	232540	6	6+80 0D55 0 S1	TC	DBB	6+80.0- 55.0	0- 4	1.5	6	9	58.9	74.743	31.5
1- 6-83	124220	1	6+80 0U15 0 S1	DM	DBB	6+80.0+ 15.0	0- 4	1.2	0	1	47.7	.032	26.5
1- 6-83	105500	6	6+90 0U15 0	TC	RV	6+90.0+ 15.0	0- 5	2.6	4	7	92.4	19.031	115.5
25- 5-83	155450	3	7+00 0D5 0 S1	TJN	RMR	7+00.0- 5.0	0- 4	2.0	14	16	62.8	116.887	48.1
27- 5-83	131440	1	7+00 0U25 0 S1	TJN	RMR	7+00.0+ 25.0	15- 30	1.7	68	85	15.8	5.956	35.3
1- 6-83	143200	1	7+10 0D35 0 S1	DM	DBB	7+10.0- 35.0	0- 4	1.1	3	5	45.7	23.145	37.9
27- 5-83	230220	3	7+10 0D15 0 S1	DPP	DBB	7+10.0- 15.0	0- 4	2.3	0	2	93.6	0.000	184.6
26- 5-83	205250	4	7+20 0D35 0 S1	SH	DBB	7+20.0- 35.0	0- 4	1.0	12	15	16.3	1.433	37.1
27- 5-83	031000	3	7+30 0D15 0 S1	DCP	DBB	7+30.0- 15.0	0- 4	2.9	9	12	118.4	33.871	149.5
1- 6-83	121210	3	7+30 0U5 0	TJN	DBB	7+30.0+ 5.0	0- 4	1.7	0	1	69.9	0.000	57.0
23- 5-83	132720	5	7+40 0D35 0 S1	MJC	DBB	7+40.0- 35.0	15- 30	.9	11	29	6.7	.784	11.4
1- 6-83	134350	6	7+50 0D25 0 S1	TJN	DBB	7+50.0- 25.0	4- 15	1.4	0	8	20.1	86.741	53.7
1- 6-83	214630	6	7+50 0D25 0 S1	TC	RV	7+50.0- 25.0	4- 30	1.7	49	67	67.8	67.941	73.9
26- 5-83	213920	3	7+50 0D15 0 S1	DP	RCG	7+50.0- 15.0	4- 15	1.4	14	23	35.6	25.944	114.8
31- 5-83	105930	3	7+50 0U15 0 S1	TJN	DBB	7+50.0+ 15.0	0- 4	5	9	11	6.0	4.048	7.4
20- 5-83	183220	2	7+60 0U15 0 S1	DM	RMR	7+60.0+ 15.0	0- 4	3.2	13	15	82.7	30.947	53.9
23- 5-83	121600	5	7+60 0U15 0 S1	TN	DBB	7+60.0+ 15.0	15- 30	1.3	13	30	14.5	3.861	44.4
1- 6-83	200430	1	7+60 0U25 0 S1	DP	RV	7+60.0+ 25.0	0- 5	1.9	15	18	70.7	110.238	39.4
1- 6-83	221310	1	7+60 0U35 0 S1	DP	RV	7+60.0+ 35.0	0- 5	3.6	23	34	69.8	85.015	62.1
19-05-83	152420	6	7+90 0D35 0 S1	MJC	RMR	7+90.0- 35.0	0- 4	1.0	6	9	11.1	4.777	13.4
26- 5-83	232020	6	7+90 0D5 0 S1	TBC	RCG	7+90.0- 5.0	0- 4	1.9	78	81	35.0	36.892	74.8
23- 5-83	155010	3	7+90 0U15 0 S1	MRS	DBB	7+90.0+ 15.0	15- 30	1.1	327	344	7.7	1.810	26.3
23- 5-83	102540	3	7+90 0U15 0 S1	MD	DBB	7+90.0+ 15.0	4- 15	2.0	14	22	58.1	11.934	145.1
26- 5-83	143930	4	7+90 0U15 0 S1	DM	STO	7+90.0+ 15.0	0- 4	6	80	83	9.2	10.729	48.9
1- 6-83	113350	1	8+00 0D35 0	MRS	DBB	8+00.0- 35.0	4- 12	1.4	1	8	27.8	8.318	34.8
24- 5-83	110400	6	8+00 0U15 0 S1	DM	RMR	8+00.0+ 15.0	0- 4	1.2	1	3	12.8	19.271	1.3
25- 5-83	161730	1	8+00 0U15 0 S1	TN	RMR	8+00.0+ 15.0	15- 30	.9	17	34	6.2	2.208	18.7
25-05-83	200640	1	8+00 0U15 0 S1	TC	RCG	8+00.0+ 15.0	4- 15	2.4	17	26	95.8	92.992	138.0
31- 5-83	122240	1	8+00 0U15 0 S1	DM	DBB	8+00.0+ 15.0	0- 4	2	7	9	2.2	.911	1.8
27- 5-83	144700	0	8+00 0U25 0 S1	TJN	RMR	8+00.0+ 25.0	4- 30	1.5	6	9	43.7	88.544	136.2
27- 5-83	233430	3	8+00 0U25 0 S1	DPP	DBB	8+00.0+ 25.0	0- 4	1.9	27	30	30.8	4.883	3.5
1- 6-83	212610	3	8+00 0U35 0 S1	SP	RV	8+00.0+ 35.0	0- 5	2.7	10	13	110.7	24.693	62.1
1- 6-83	181630	1	8+10 0D45 0 S1	TBC	DBB	8+10.0- 45.0	0- 4	4	19	31	16.9	9.004	28.5
19-05-83	182130	6	8+10 0D35 0 S1	MJC	RMR	8+10.0- 35.0	0- 4	8	14	16	9.2	8.497	34.5
27- 5-83	212520	6	8+10 0D35 0 S1	TCC	DBB	8+10.0- 35.0	0- 4	1.9	0	2	73.1	11.897	29.4
24- 5-83	142810	6	8+10 0D5 0 S1	DM	RMR	8+10.0- 5.0	4- 15	1.2	12	21	8.3	2.949	42.5
19-05-83	184250	2	8+10 0U15 0 S1	MJC	DBB	8+10.0+ 15.0	0- 4	2.6	7	10	41.7	77.442	46.3
24- 5-83	185500	1	8+20 0D35 0 S1	DP	RCG	8+20.0- 35.0	0- 5	1.0	13	16	6.9	.395	1.4
31- 5-83	174600	6	8+20 0U25 0	TJN	DBB	8+20.0+ 25.0	15- 30	1.8	37	44	55.3	50.580	133.1
31- 5-83	184900	6	8+20.0U25 0	TC	RMR	8+20.0+ 25.0	4- 15	2.2	8	17	86.8	131.385	79.2
23- 5-83	115130	3	8+20 0U35 0 S1	TJN	RMR	8+20.0+ 35.0	0- 4	2.6	10	13	106.7	109.506	93.7
20- 5-83	110130	6	8+30 0D25 0 S1	TC	RMR	8+30.0- 25.0	15- 30	1.0	27	45	6.9	12.875	31.5
26- 5-83	192450	1	8+30 0D15 0 S1	TJN	RMR	8+30.0- 15.0	15- 30	2.3	18	36	37.1	9.819	318.7
26- 5-83	224640	1	8+30 0D15 0 S1	TBC	DBB	8+30.0- 15.0	4- 15	1.4	77	85	34.7	29.893	51.2
27- 5-83	202300	6	8+30 0D15 0 S1	MRS	DBB	8+30.0- 15.0	0- 4	1.9	14	16	31.5	110.730	60.8

Figure 8-15. - Typical phase II grout summary report (sheet 1 of 2).

RIDGWAY DAM, STAGE II
GROUT SUMMARY DATA - DATE 10-10-83
SORTED BY LOCATION

HAYWARD BAKER COMPANY
GROUTING CONTRACTOR'S NO: 1194
GRANITE CONSTRUCTION COMPANY
GENERAL CONTRACTOR'S NO: XXXX

TOTAL NUMBER OF BAGS CEMENT	TOTAL WATER (CU FT)	TOTAL TIME (MIN)	AVG VOL GROUT INJECTED /FOOT (CF/LF)	AVG NO BAGS INJECTED /FOOT (BG/LF)	DATE	TIME	LOCATION	STAGE DEPTH	DESIGNATION
7 20	25 04	32	0 00	0 000	27- 5-83	233000	5+79 0- 23 0	0- 4	5+79 0023 0 S1
74	3 73	32	0 00	0 000	27- 5-83	150630	5+83 3- 13 0	0- 4	5+83 0013 0 S1
12 51	28 74	33	0 00	0 000	28- 5-83	032300	5+83 3- 13 0	4- 10	5+83 0013 0 S1
29 52	88 01	103	0 00	0 000	27- 5-83	195300	6+00 0- 25 0	0- 4	6+00 0025 0 S1
23 92	85 15	70	0 00	0 000	28- 5-83	200300	6+03 0- 33 0	0- 4	6+03 0033 0 S1
65 34	207 83	424	21 32	4 354	31- 5-83	194300	6+30 0- 03 0	15- 30	6+30 0003 0
7 66	18 82	12	0 00	0 000	28- 5-83	231100	6+30 0- 45 0	0- 4	6+30 0043 0 S1
13 69	37 57	33	0 00	0 000	27- 5-83	205110	6+32 0- 23 0	0- 4	6+32 0023 0 S1
15	1 18	6	00	010	31- 5-83	202010	6+60 0- 20 0	15- 30	6+60 0020 0
84 27	291 99	213	30 29	7 661	1- 6-83	231100	6+60 0- 35 0	4- 15	6+60 0035 0
0 01	27 19	28	0 00	0 000	27- 5-83	232540	6+80 0- 55 0	0- 4	6+80 0055 0 S1
10 15	21 69	28	0 00	0 000	1- 6-83	120230	6+80 0+ 13 0	0- 4	6+80 0013 0 S1
51 24	90 61	86	0 00	0 000	1- 6-83	195500	6+90 0+ 15 0	0- 5	6+90 0015 0
10 30	35 87	30	0 00	0 000	25- 5-83	135650	7+00 0- 5 0	0- 4	7+00 005 0 S1
5 43	32 57	78	2 25	342	27- 5-83	181400	7+00 0+ 25 0	15- 30	7+00 0025 0 S1
21 49	32 28	46	0 00	0 000	1- 6-83	143200	7+10 0- 23 0	0- 4	7+10 0035 0 S1
25 01	87 47	75	0 00	0 000	27- 5-83	230130	7+10 0- 15 0	0- 4	7+10 0015 0 S1
9 59	32 37	56	0 00	0 000	26- 5-83	205250	7+30 0- 23 0	0- 4	7+30 0035 0 S1
63 96	120 00	150	0 00	0 000	27- 5-83	031000	7+30 0- 15 0	0- 4	7+30 0015 0 S1
23 83	65 39	78	0 00	0 000	1- 6-83	121210	7+30 0+ 5 0	0- 4	7+30 005 0
1 35	10 77	33	74	090	23- 5-83	132720	7+40 0- 35 0	15- 30	7+40 0035 0 S1
13 69	47 87	102	4 09	1 240	1- 6-83	124350	7+50 0- 25 0	4- 15	7+50 0025 0 S1
19 60	69 36	69	2 84	756	1- 6-83	214630	7+50 0- 25 0	4- 30	7+50 0025 0 S1
18 56	103 71	160	10 43	1 687	26- 5-83	212930	7+50 0- 15 0	4- 15	7+50 0015 0 S1
1 36	6 78	22	0 00	0 000	31- 5-83	105930	7+50 0+ 15 0	0- 4	7+50 0015 0 S1
21 33	63 63	69	0 00	0 000	28- 5-83	103320	7+60 0+ 15 0	0- 4	7+60 0015 0 S1
8 00	40 60	95	2 96	339	23- 5-83	121600	7+60 0+ 15 0	15- 30	7+60 0015 0 S1
11 46	33 77	31	0 00	0 000	1- 6-83	200630	7+60 0+ 25 0	0- 5	7+60 0025 0 S1
19 77	52 91	71	0 00	0 000	1- 6-83	221310	7+60 0+ 35 0	0- 5	7+60 0035 0 S1
2 63	12 17	44	0 00	0 000	19-05-83	152620	7+90 0- 35 0	0- 4	7+90 0035 0 S1
21 41	65 92	123	0 00	0 000	26- 5-83	222020	7+90 0- 5 0	0- 4	7+90 005 0 S1
3 09	24 75	76	1 75	406	23- 5-83	155010	7+90 0+ 15 0	15- 30	7+90 0015 0 S1
34 66	132 32	95	13 19	3 271	23- 5-83	182540	7+90 0+ 15 0	4- 15	7+90 0015 0 S1
13 29	62 85	164	0 00	0 000	26- 5-83	143930	7+90 0+ 15 0	0- 4	7+90 0015 0 S1
10 00	29 86	51	4 35	1 260	1- 6-83	113350	8+00 0- 35 0	4- 11	8+00 0035 0
23	1 16	4	0 00	0 000	26- 5-83	110400	8+00 0+ 15 0	0- 4	8+00 0015 0 S1
3 20	17 59	71	1 26	147	25- 5-83	161730	8+00 0+ 15 0	15- 30	8+00 0015 0 S1
37 97	119 20	109	12 59	3 653	25-05-83	200640	8+00 0+ 15 0	4- 15	8+00 0015 0 S1
19	92	11	0 00	0 000	31- 5-83	132240	8+00 0+ 15 0	0- 4	8+00 0015 0 S1
42 23	115 38	149	5 24	1 620	27- 5-83	144700	8+00 0+ 25 0	4- 30	8+00 0025 0 S1
69	3 18	10	0 00	0 000	27- 5-83	233630	8+00 0+ 25 0	0- 4	8+00 0025 0 S1
22 38	51 15	127	1 00	0 000	1- 6-83	212610	8+00 0+ 35 0	0- 5	8+00 0035 0 S1
5 76	17 69	146	0 00	0 000	1- 6-83	181630	8+10 0- 45 0	0- 4	8+10 0045 0 S1
6 55	31 29	114	0 00	0 000	19-05-83	182130	8+10 0- 35 0	0- 4	8+10 0035 0 S1
7 65	24 67	34	0 00	0 000	27- 5-83	212320	8+10 0- 25 0	0- 4	8+10 0025 0 S1
7 36	56 92	147	3 66	678	26- 5-83	142810	8+10 0- 5 0	4- 15	8+10 005 0 S1
17 85	37 91	44	0 00	0 000	19-05-83	184230	8+10 0+ 15 0	0- 4	8+10 0015 0 S1
14	1 18	4	0 00	0 000	26- 5-83	183500	8+20 0- 35 0	0- 5	8+20 0035 0 S1
65 50	110 67	129	8 87	3 034	31- 5-83	174600	8+20 0+ 25 0	15- 30	8+20 0025 0
19 61	69 69	66	7 20	1 761	31- 5-83	184900	8+20 0+ 25 0	4- 15	8+20 0025 0
26 81	79 56	70	0 00	0 000	25- 5-83	115120	8+20 0+ 35 0	0- 4	8+20 0035 0 S1
2 56	20 29	71	1 66	169	20- 5-83	110130	8+30 0- 25 0	15- 30	8+30 0025 0 S1
24 47	204 74	300	14 58	1 631	26- 5-83	192450	8+30 0- 15 0	15- 30	8+30 0015 0 S1
13 39	66 24	72	4 63	1 236	26- 5-83	224640	8+30 0- 15 0	4- 15	8+30 0015 0 S1
17 14	52 44	82	0 00	0 000	27- 5-83	002300	8+30 0- 15 0	0- 4	8+30 0015 0 S1

Figure 8-15. - Typical phase II grout summary report (sheet 2 of 2).

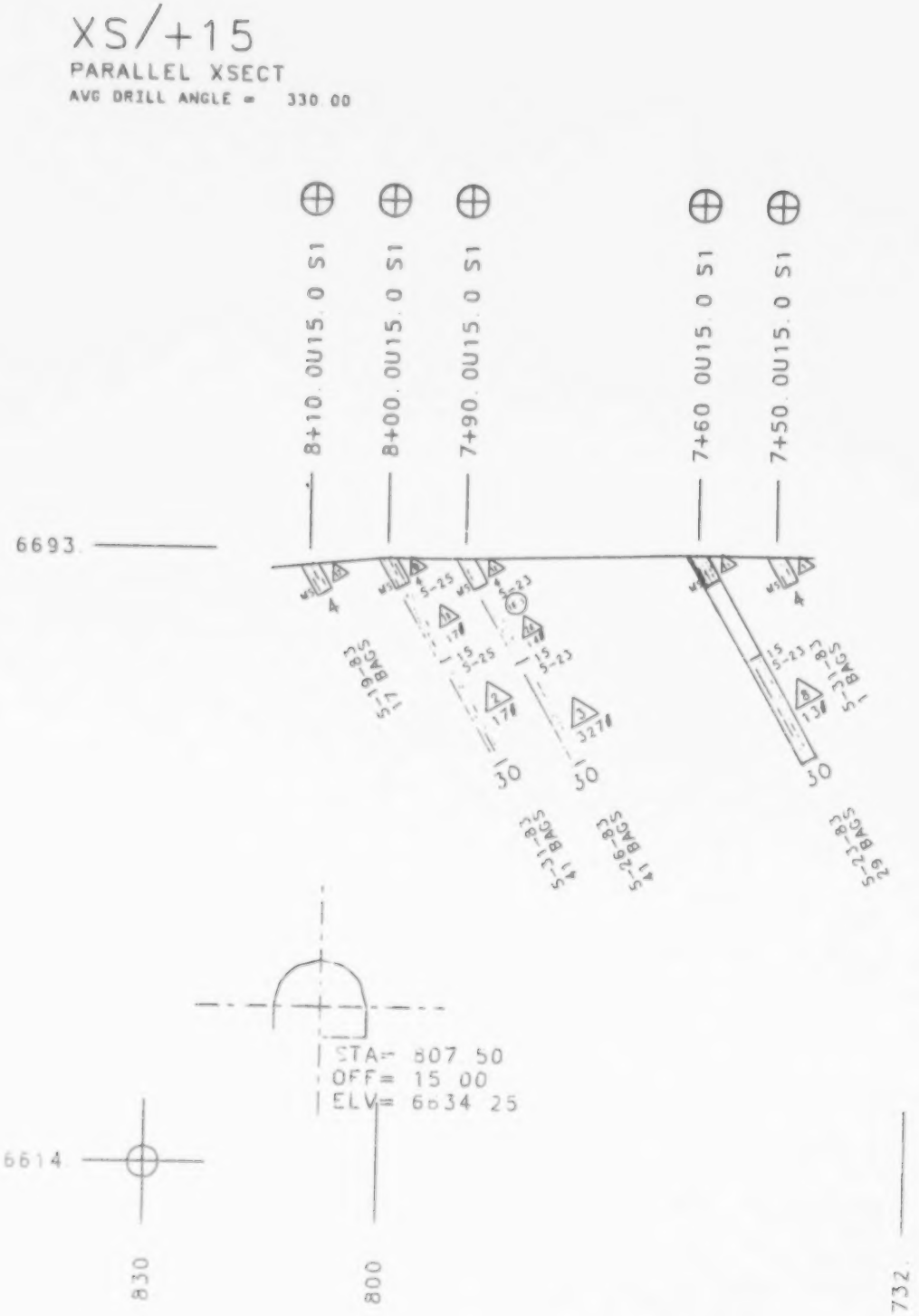


Figure 8-16. - Typical longitudinal plot of grout holes.

The central plant and monitoring trailer also allowed Bureau inspectors to quickly evaluate the status of grouting at each of the holes without traversing the entire site.

Extremely important to an efficient grouting operation are the control center personnel who schedule the grouting operations from the monitoring trailer. This position requires a person who is interested in grouting, able to schedule operations, and who can deal tactfully with the contractor and the owner. It is anticipated that a short training course will be developed for employees in this critical position.

All Bureau inspectors who were interviewed liked the electronic monitoring system and felt that it decreased the grouting time. This is important because many large construction claims result from unfinished grouting that delays embankment placement.

The introduction and use of electronic monitoring has not been without problems, however. These problems have been primarily caused by four things: (1) line-voltage variations, (2) microcomputer malfunctions, (3) software problems, and (4) internal resistance to change. Variations in line voltage caused by the sudden startup of electrical equipment resulted in the loss of hole data on many occasions. This problem was solved by supplying a constant line-voltage source. Microcomputer malfunctions caused by dust filtering into the trailer required the use of a humidifier, an electrostatic precipitator, and an air conditioner. Only one type of diskette proved to be usable in the microcomputer; other diskettes lost or would not store data. Later, it was found that electronic problems with the microprocessor main boards had created many of the problems. Numerous minor problems were encountered in the real-time plotting and hard copy operations because of the long plotting time required for each graph. This was especially bothersome during the plotting of all six graphs after completion of the stage. At these times as much as 1 hour was needed to plot all graphs on long duration holes. This long plotting time was caused by the short scanning time used to update the flow and pressure data for each hole. It was also discovered that data being transmitted from other holes in progress was being lost during plotting operations. Most of these problems have been solved by reprogramming.

Another of the major problems encountered concerned the length of time to continue pumping a given mix. Although this question has not yet been answered, it is clear that the time spent pumping a given mix can be reduced substantially using electronic monitoring.

When the phase II software and plotting became available in August 1983, problems were experienced with the plotter unit and the software. Considerable time was spent correcting these problems. Additional problems resulted from a misevaluation of the time it would take to update and correct the phase I data and store it in the phase II system. As a result of this misevaluation the phase II equipment was not fully utilized in real-time operations until late November 1983.

Preliminary evaluation of the phase I plots from the right abutment grouting and the discussions with field personnel indicate the following:

1. Plots of flow versus time and collar pressure versus time have proven extremely valuable in evaluating grouting operations, in denoting anomalous grouting conditions, and in confirming hole refusal.

2. Plots of flow divided by pressure versus time were not useful in evaluating blanket hole grouting takes because of numerous near-surface leaks.

3. Plots of bags per hour versus time were useful in determining when refusal criteria were about to be met.

There is little doubt that electronic monitoring of pressure grouting will bring a reevaluation of many of the standard concepts and criteria held by grouters over the last 20 to 30 years. For example, as a result of the current data, the Bureau has begun to question its refusal criteria. Yet it is clear that some problems must be overcome; not the least of which is the belief, of many persons, that the method has little merit. Grouting has been, in many senses, a "black art", subject to strong personal opinions but limited real data. However, with this new procedure the individuality of each site can be quickly established and much additional information can be provided to more observers. Grouting judgements can then be based on real-time plots and instantaneous data summaries.

The electronic monitoring method, when combined with accoustical 3-D location of grout travel, which is also being investigated at the Ridgway Dam site [4], may help provide some answers to the question of where subsurface grout travel occurs.

FUTURE WORK

Electronic monitoring of grouting is scheduled for two more major dam projects: (1) Upper Stillwater Dam, a roller-compacted concrete dam, and (2) Brantley Dam, a composite earthfill-gravity concrete dam. The specifications for electronic monitoring at Upper Stillwater Dam have been revised significantly to reflect the items learned at Ridgway Dam. An evaluation of the total program at Ridgway Dam is planned following completion of the grouting. These results will be published at the earliest possible date.

ACKNOWLEDGMENTS

The writer acknowledges Wallace H. Baker for his critical comments and for the creative decision to forcefully pursue the advancement of cement grouting procedures; and Bob Reinhardt, Jenith Stodolski, and Donna Almond, at the Ridgeway Dam project site, for their candid assessments of the electronic monitoring procedures and the revisions needed.

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APPENDIX A

Requirements for Microcomputer Recording and Presentation of Grouting Data

1. During water testing, the following information will be digitally displayed, real-time:

   a. Time for current injection (min)
   b. Total volume injected (gal)
   c. Flow rate of water injection (gal/min)
   d. Water pressure (lb/in^2)

2. During water testing, any one of the following plots could be displayed on a video monitor; each plot would have to be called up separately:

   a. Water pressure (lb/in^2) versus time (min)
   b. Flow volume (gal) versus time (min)
   c. Flow divided by pressure versus time (min) (b/a)

3. Following completion of water testing, a summary of the water testing data will be made to include:

   a. Final plots of flow (gal/min), volume (gal), pressure (lb/in^2), and flow (gal/min) divided by pressure (lb/in^2) versus time (min)
   b. Maximum and minimum flows and pressures
   c. Average flow rate and total volume of water injected
   d. Permeability of the stage will be calculated using formulas in Designation E-18 of the "Earth Manual" (ft/yr) and in lugeons
   e. Depths at which partial or total water losses took place

4. A tabular listing of routine drilling and grouting information for each grout hole wll be kept in computer storage for recall at any time. This information will consist of:

   a. Name of dam
   b. Project name
   c. Specifications numbers
   d. Day and date
   e. Shift
   f. Inspector
   g. Curtain hole (right or left abutment or tunnel) or blanket hole (right or left abutment)
   h. Hole designation
   i. Vertical or angled (give degrees right or left from vertical, looking downstream)
   j. Driller or technician
   k. Percussion or rotary
   l. Primary, secondary, etc.
   m. Elevation at top of hole (ft)

n. Top and bottom depth of stage being grouted (ft measured along hole)
o. Location (station and offset from dam centerline)

5. The real-time grouting information displayed on the video monitor will be as follows:

a. Hole
b. Location
c. Inspector
d. Technician
e. Stage depth interval
f. Total time hole injected (min)
g. Injection time current water/cement ratio (min)
h. Current water/cement ratio
i. Total volume grout mix injected (ft^3)
j. Total bags injected
k. Flow rate grout mix injection (ft^3)
l. Flow rate (bags per hour)
m. Injection pressure at collar (lb/in^2)
n. Injection pressure at bottom of stage (lb/in^2)
o. Flow rate divided by pressure

The data recorded should be equivalent to that obtained by using Form 1374: Grouting Inspector's Report. A hard copy printout of this information should be obtainable.

6. Video monitor plots of the following variables can be made at all times that grouting is proceeding on any hole:

a. Total volume of grout mix pumped in this stage versus time
b. Pressure (lb/in^2) versus time (min)
c. Flow (ft^3 of mix) versus time (min)
d. Flow (ft^3 of mix) divided by pressure (lb/in^2) plotted versus time (min)
e. Bags per hour versus time (min)
f. Cubic feet of cement versus time (min)

7. System operators will be able to input comments, referenced to a given time during the injection process. These comments should be capable of being obtained as a sequential list, referenced to the stage and hole, and will be input only during the actual grouting.

8. Following completion of grouting on any stage, a summary for that stage will be made, including the following information:

a. Final plots of all variables shown in section 6
b. Maximum and minimum values for:

(1) Flow of grout mix (ft^3/min)
(2) Flow of cement (bags per hour)
(3) Flow divided by pressure
(4) Injection pressure at collar (lb/in^2)
(5) Injection pressure at bottom of stage (lb/in^2)

c. Total volume of grout injected, including total solids injected and total water injected
d. Average volume of grout injected per foot of hole and average number of bags injected per foot of hole
e. Sequential listing of all comments
f. Incorporation of the above information excluding plots and comments in computer storage, along with the water testing summary for the stage in question, for future reference

9. A data summary should be kept for each of the primary, secondary, tertiary, etc. holes, which includes the following:

a. Lineal feet of drilling
b. Total hours drilled
c. Lineal feet drilled per hour (a/b)
d. Take (ft^3) of cement
e. Take (ft^3) of solids per linear foot of hole (d/a)
f. Total hours grouted
g. Ft^3 of cement per hour (d/f)

This summary should be kept for each month and for the total grouting performed to date. Blanket grouting data should be kept separately from curtain grouting data. The summary for percussion-drilled blanket holes should be kept separately from that of rotary-drilled blanket holes.

10. The system shall have the ability to plot profile plots of the ground surface and the location and orientation of the grout holes for a given area on 21- by 36-inch sheets, and in a bounded area 19-1/2- by 22-1/2-inches. Our standard scale for plotting on these sheets has been 1 inch equals 20 feet, both horizontally and vertically. Each stage grouted should be shown on the plot, with the following items referenced to that stage:

a. Date grouted
b. Maximum water pressure in lb/in^2 used to test the stage
c. Total water take in ft^3
d. Grouting pressure in lb/in^2
e. Total grout take in bags
f. Locations of partial and total water losses

At the bottom of the plotted hole the depth of the hole and the total bags injected should be recorded. The top and bottom depth of each stage should be printed, in feet, at the side of the hole.

11. Hard copy plots of all video displays should be available.

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